

**FOUNDATION INVESTIGATION  
AND DESIGN REPORT  
HIGHWAY 69, FOUR-LANING  
FROM 4KM SOUTH OF ESTAIRE TO 1KM NORTH OF HIGHWAY 537, 12KM  
CN RAIL OVERHEAD STRUCTURES AND APPROACH EMBANKMENTS  
ONTARIO**

**G.W.P. 312-99-00,  
WP 5047-00-01, SITE 46-494N  
WP 5048-00-01, SITE 46-494S  
Geocres Number: 41I-184  
VOLUME 1/2**

**Report to**

**Ministry of Transportation Ontario  
Planning and Design Section, North Bay**

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**PART 1: FACTUAL INFORMATION**

**1 INTRODUCTION**

The proposed widening and realignment of Highway 69 over a 12km length, from 4km south of Estaire to 1km north of Highway 537 will include the crossing of a low-lying area that is traversed by a rail track owned and operated by CN Rail. The site is located approximately 10km south of Sudbury, just south of the intersection of the existing Highway 69 and Highway 537, at the boundary between the Townships of Secord and Dill.

The site area is characterized by a swamp close to CN Rail's track, and deep compressible deposits. The design will include the construction of two parallel overhead structures over the swamp and rail tracks and associated approach embankments.

This report presents the factual findings obtained from a foundation investigation for the proposed overhead structures, referred herein as Southbound Lane (SBL) and Northbound Lane (NBL) bridges and approach embankments north of the bridges.

Peto MacCallum (Peto) carried out a preliminary investigation in the spring of 2003 at the site and the factual data from that investigation was available for the current assignment.

The purpose of the current investigation presented herein was to explore the subsurface conditions at the proposed bridge sites and approach embankments and, based on the data obtained, to provide a borehole location plan for each bridge, borehole logs, stratigraphic profiles and cross-sections and a written description of the subsurface conditions. A model of the subsurface conditions was

developed through considering a combination of the data from the previous Peto report and the data obtained in the course of the present investigation.

Thurber carried out the investigation as a consultant to Ministry of Transportation Ontario (MTO) under the Agreement Number 5005-A-000409.

## **2 SITE DESCRIPTION**

The realigned Highway 69 at the proposed bridge sites runs in an approximate south-north direction, parallel to and 300m east of the existing Highway 69.

The general site area is located within the physiographic region known as the Canadian Shield, locally characterized by Pre-Cambrian bedrock of the Central Gneiss Belt. The bedrock elevation varies significantly in the area. Bedrock outcrops are present at the site but the bedrock is mostly overlain by glacial deposits (till) with particle sizes ranging from silt to boulder size, deep post-glacial lake sediments (glaciolacustrine origin) and by organic deposits in the low lying areas.

The proposed highway alignment will traverse a 450m wide depression/channel with bedrock outcrops to the north and south. CN Rail track of the Bala Subdivision crosses the channel in an east-west orientation. Drainage in the general area is to the southwest and is generally controlled by the Wanapitei River, north of the site.

The southern portion of the channel, within 100m of the south end of the channel, is occupied by a low-lying, flat, wet and poorly drained swamp. Standing water was noted in the swamp at several locations. CN Rail track crosses this swamp. The area to the south of the swamp consists of a thin veneer of organic soil over bedrock or bedrock outcrops. The terrain is rugged and sparsely covered with mature vegetation and trees.

The ground rises to the north with a vertical relief of up to 5m above the swamp elevation. An area extending about 100m north of the swamp is wooded, consisting mostly of mature spruce and cedar, beyond which there is farmland.

## **3 SITE INVESTIGATION AND FIELD TESTING**

The site investigation and field testing discussed herein was carried out between May 11 and June 8, 2004. The site investigation consisted of:

- Test pitting at five locations at each of the south abutment foundation elements for the proposed SBL and the NBL bridges. The test pits were labelled as BH-1S through BH-5S and BH-1N through BH-5N for the SBL and NBL bridges, respectively. These test pits were advanced with a hand shovel and encountered bedrock within 0.4m of surface. In addition, the exposed bedrock in the proximity of the abutments was mapped for structural features and weathering. The original program included coring bedrock at two locations at each abutment. In light of the fact that massive bedrock was present at the surface at the south abutment locations, it was decided in discussion with MTO foundations that coring of bedrock would

not be required. Instead, test pitting and a detailed surface inspection of the bedrock were carried out.

- Drilling and sampling nineteen boreholes at the bridge foundation elements. Boreholes BH-6S through BH-16S and BH-6N through BH-13N located at the proposed piers and north abutment locations of the SBL and NBL bridges. These boreholes were drilled to depths of 3.6m to 54.6m at the SBL Bridge site and 13.1m to 52.1m at the NBL Bridge site. These boreholes were advanced to obtain information relating to the subsurface conditions for the design of the overpass structure foundation elements.
- Drilling and sampling four boreholes, Boreholes BH-17S, BH-18S, BH-14N and BH-15N at the centreline of the north approach embankments to the bridges. These boreholes were drilled in order to provide information for the design of the approach embankments to the bridge. The boreholes were advanced to refusal encountered at depths of 46.0m to 51.3m.
- Advancing eight piezocone tests (CPTU-1S through CPTU-4S and CPTU-1N through CPTU-4N) under the north approach embankments to depths ranging from 37.9m to 52.7m to provide more detailed information on the stratigraphy and consolidation characteristics of the compressible foundation soils for the design of the approach embankments.

The locations of the test holes are shown on the attached Drawing No.15-64-15-1 – Test Hole Location Plan. The test hole coordinates and elevations are summarized in attached Sheets 1, 3, 5 and 6.

Surveyors from Sutcliffe, Rody Quesnell Inc. marked the borehole locations in the field and utility clearances were obtained by Thurber prior to any drilling being carried out. Authorization to enter the properties and to carry out the investigation program was obtained by Thurber from the following land owners:

- Canadian National Railway
- INCO Limited, Ontario Operations
- Heino and Annikki Pennanen

Preparation of access to the borehole locations was carried out under Thurber's supervision by Interpaving Ltd. of Sudbury. Access preparation included construction of a temporary road from Hwy 69 to the south abutment locations. In addition, 80 blasting mats were used to provide access to the drill rigs from the south abutment locations to drilling sites in the swamp.

George Downing Estate Drilling Ltd. supplied and operated the drilling and sampling equipment, mobilizing from Calumet, Quebec. Two track mounted drill rigs were used concurrently during the investigation program to meet the project schedule.

A combination of hollow stem auger and mud rotary drilling techniques were used to advance the boreholes and samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT), thin wall Shelby tubes and diamond core barrels. Standpipe piezometers were installed at the foundation element locations except at the south abutments and Pier 1S, where the bedrock was either visible at surface or shallow in the

swamp. Standpipe installation consisted of 19mm PVC pipe slotted at the bottom over 1.5m to 3.0m length. The slotted pipe was surrounded by M3 Sand and sealed in place a bentonite plug with minimum thickness of 0.6m. The remainder of the hole was grouted using a cement-bentonite grout in accordance to O'Reg 128. Additional standpipe installation details are provided in the borehole logs in Appendix A.

ConeTec Inc. supplied and operated the piezocone testing, mobilizing from Vancouver, British Columbia. The piezocone testing included standard static penetration with pore pressure measurement and pore pressure dissipation tests at selected depths in order to obtain horizontal coefficient of consolidation values for the design of wick drains. In addition pore pressure dissipation tests were carried out in the sand layer underlying the silty clay deposit to obtain the piezometric head and the possible presence of artesian pressure at that elevation. Seismic cone testing was carried in CPTU-1S and CPTU-1N. A summary of the piezocone locations and results of the pore pressure dissipation tests are provided in ConeTec's report included in Appendix D.

A member of Thurber's engineering staff supervised the piezocone testing, drilling and sampling operations on a full time basis. The inspector logged the boreholes and the recovered samples and processed them for transport to Thurber's Oakville office. All boreholes were grouted on completion of the drilling program in accordance to O'Reg 128. All blasting mats were removed from the site after completion of drilling.

#### **4 LABORATORY TESTING**

All recovered soil samples were subjected to visual identification and natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A.

Selected samples were subjected to gradation analysis (sieve and hydrometer), Atterberg Limit, Oedometer and Triaxial testing. The test results are shown on the Record of Borehole sheets in Appendix A and on the charts in Appendix B.

#### **5 DESCRIPTION OF SUBSURFACE CONDITIONS**

##### **5.1 General**

Reference is made to the Record of Borehole sheets for boreholes drilled by Thurber in Appendix A, to the Record of Borehole sheets prepared by Peto MacCallum Ltd. (Peto) included in a report dated May 23, 2003 and to ConeTec's report included in Appendix D. Details of the encountered soil and rock stratigraphy are presented in these appendices and on the attached drawings Sheets 1 through 6. A generalized description of the stratigraphy is given in the following paragraphs. The factual information at the borehole locations governs any interpretation of site conditions.

In general terms, the site was found to be underlain by granitic bedrock of the Canadian Shield. The bedrock is overlain by broadly graded generally compact to dense granular soils including silt, sand, cobbles and boulders, which are overlain by compressible glaciolacustrine and glaciofluvial soils and organic soils. The glaciolacustrine deposits consist of plastic silt and clay and are overlain by outwash glaciofluvial loose to compact sand and silt with clayey material interbeds or peat deposited in the recent geological past and locally by fill at the railway track and at the boreholes drilled along the northern boundary of CN Rails' right-of-way (ROW). The compressible glaciolacustrine clay deposit is fairly thick under the north half of the bridges and under the north approach embankment with thickness up to 35m.

## **5.2 Fill, Topsoil and Peat**

The CN Rail track crosses the site on a fill approximately 1 to 2m above the swamp level. The rail embankment material was not sampled during the investigation. The boreholes drilled long the northern boundary of CN Rail's ROW, at the location of Piers 3S and 3N, encountered sand fill up to 1.5m depth.

A thin veneer of topsoil, up to 100mm thick was encountered at surface overlying bedrock in at the south abutments of both bridges. Up to 250mm of topsoil was also encountered in the higher grounds located north of Piers 3S and 3N. Topsoil was absent in the boreholes located in the low-lying areas north and south of the railway track and it was mixed with sandy and silty soils in the farmland north of the bridges. The topsoil thickness may vary between boreholes and in other areas of the site. Farmland areas may have thicker topsoil and topsoil mixed with surficial mineral soils.

A thick layer of peat was encountered in the swamp south of the CN Rail track. The peat was described as fine fibrous dark brown to black and varied in thickness from zero along the south bank of the swamp where the bedrock is shallow, up to 5.8m at Pier 2N (BH-8N), south of CN Rail track. North of CN Rail track, up to 0.2m of peat was encountered in two boreholes (612-11S and BH-13S) close to Pier 3S. The peat was very loose and offered low resistance to penetration of the SPT split spoon sampler which sank under the weight of the SPT hammer or rods.

## **5.3 Sand/Silt and Clayey Silt to Silty Clay Interbed Layers**

An outwash glaciofluvial deposit was encountered north of the CN Rail track at surface or underlying a thin layer of topsoil. The deposit consists mainly of interbedded layers of non-plastic silt and sand and locally by clayey layers of variable thickness. The thickness of the deposit typically ranged from 7 to 14m, except close to the exposed bedrock at the north end of the approach embankments where it is absent. The elevation of the bottom of this deposit typically ranged from EL.210 and EL.215. The SPT "N" blow counts in the sand/silt deposit ranged from 2 to 45 with an average value of 13 and most values ranging from 10 to 20. Based on the SPT values noted above the sand/silt layer is described as loose to dense and generally compact. Locally high blow counts, in excess of 50, indicate

the presence of obstruction to the split spoon penetration probably due to the presence of cobbles and boulders.

A higher clay content material, ranging from some clay to silty clay was encountered within the sand/silt deposit in some of the test holes, between approximate EL 215 and EL.220. The clay content within this material appears to increase towards northwest, in particular north of Station 10+620, where it is characterized as low plastic to non-plastic, clayey silt to silty clay. The thickness of the clayey silt to silty clay layer typically ranged from 2, to 5m. The SPT "N" blow counts in the clayey silt to silty clay layer ranged from 2 to 8 with an average value of 6. In-situ vane and piezocone tests indicated undrained shear strength values ranging from 20kPa to 50kPa, reflecting consistency of soft to firm.

Gradation test results carried out on samples collected from the sand/silt deposit are presented in Figures B11, B2, B20 and B21, Appendix B. The test results indicate that the sand/silt deposit is composed of a broad range of sand to clay particle sizes ranging from 0% to 66% sand, 65% to 90% silt and 5% to 15% clay.

The natural moisture content of this deposit ranges from 20% to 30%. The higher values are related to higher clay contents.

#### 5.4 Silty Clay

A thick deposit of silty clay was encountered in most boreholes and piezocones underlying the peat and the sand/silt deposits. The thickness of the deposit ranged from zero at the south end of the swamp, and at the boundary of the Secord and Dill Townships, to a maximum thickness of more than 30m between Piers 3S and 3N and Station 10+640 at the underneath the approach embankments. The elevation of the top of this deposit ranges from approximate EL.220 at the north and south boundaries of the deposit to about EL.210 at the location of the north abutments. The lowest elevation of the bottom of this deposit was at approximate EL.182 also at the north abutments.

A series of Atterberg Limit tests, summarized in Figures B1 through B10 in Appendix B, indicated that the silty clay deposit has variable plasticity and is classified as follows, according to the Modified Unified Soil Classification System:

- Low plastic silty clay (CL) in the upper portion of the deposit. The lowest elevation of the base of the CL material is at approximate EL.200 at the north abutment locations and it rises to approximately EL.215 at the southern and northern boundaries of the deposit.
- Intermediate and high plastic silty clay (CI-CH) below the CL material. This material extends above approximate EL.185 to EL.187. This material was described as varved at and north of the north abutments.
- Intermediate and low plastic silty clay (CL-CI) below approximate EL.185.

Hydrometer tests carried out on silty clay samples, summarized on Figures B15 through B19 and Figure B23 through B26 in Appendix B, indicate that the clay content varies with plasticity as follows:

- Low plastic silty clay (CL): 30% to 45% clay content
- Intermediate plastic silty clay (CI): 46% to 64% clay content
- High plastic silty clay (CH): 68% to 86% clay content

The water content values for the upper and lower portions of the silty clay deposit, referred herein as CL and CL-CI, respectively, ranged from 25% to 53%, with most values varying between 33% and 37%. The intermediate and high plastic silty clay (CI-CH) had moisture content values ranging from 22% to 60% with most values from 37% to 46%.

Oedometer tests were carried out on four samples. Two of the samples were retrieved from the upper portion of the silty clay deposit (CL) and two samples were retrieved from the CI-CH deposit. The test results are presented in Appendix B and summarized in attached Table 5.1.

Consolidated Undrained (CU) triaxial tests were carried out on two samples of the clay. Details of the triaxial tests are provided in Appendix B and a summary of the results are provided in Table 5.2.

Undrained shear strength ( $S_u$ ) inferred from in situ vane and the piezocone tests carried out in the upper (CL) and the lower (CL-CI) silty clay deposits ranged from 15kPa to 100kPa, with an average value of 55kPa. This deposit can be generally described as soft to very stiff.

Undrained shear strength ( $S_u$ ) inferred from in situ vane and the piezocone tests carried out in the intermediate silty clay (CI-CH) deposit ranged from 25kPa to 100kPa, with an average value of 85kPa. This deposit can be generally described as soft to very stiff.

In situ vane tests carried out in the silty clay deposit located south of the CN Rail's track showed that within 5m to 10m below the bottom of the deposit of peat, above EL.205, the material was very sensitive, with sensitivity values ( $S_r$ : ratio of peak and remoulded undrained shear strength) ranging from 8 to 11. Elsewhere the silty clay deposit was moderately sensitive to sensitive, with  $S_r$  values ranging from 2 to 8. The piezocone tests also detected the presence of sensitive clays in the north approach embankment area, particularly the upper portion of the silty clay deposit between EL.199 and EL.210.

## 5.5 Clayey Silt to Silt

A layer of clayey silt to silt was encountered below approximate EL185 to EL.187 underlying the silty clay deposit north of the north abutments. The natural water content of this material ranged from 27% to 40% and the material was characterized as CL-ML according to the Unified Soil Classification System. Hydrometer tests carried out on two samples collected from this deposit are shown of Figures B14 (BH-15S – EL.184.12) and

B22 (BH-14N - EL.184.39). The silt/clay contents in the samples tested were 75%/25% and 80%/20%, respectively. This deposit was described as stiff to very stiff.

## **5.6 Silt**

A deposit of silt was encountered underlying the silty clay and clayey silt deposits in most test holes advanced north of Piers 2S and 2N. The silt was non-plastic to low plastic and it is described as ML (NP) to ML according to the Unified Soil Classification System. Hydrometer tests in this material, shown on Figures B14 and B22 indicate clay content values ranging from 10% to 15% with the remaining consisting of silt and up to 4% sand. The natural water content was uniform and approximately 25%. The SPT "N" values ranged from 1 to 48 with an average value of 26. The low values of "N" are attributed to disturbance at the bottom of the borehole caused by drilling. This material is described as compact to dense.

## **5.7 Sandy Silt, Silty Sand, Sand and Gravel**

Compact to dense sandy silt, sand and gravel was encountered overlying bedrock in three boreholes drilled at and north of the SBL bridge north abutment. One gradation test carried out on a sample collected from this deposit (Figure B13) indicated a sand/silt/clay content of 30%/61%/9%.

The bedrock at Piers 1S and 1N are overlain by loose to compact cohesionless soils with gradation ranging from silt to gravel sizes and occasional cobbles. Two of the boreholes (BH-11S and BH-8N) drilled at the locations of Piers 2S and 2N encountered a thin layer of sand and gravel, less than 1m thick, overlying bedrock. In both boreholes boulders up to 0.4m in vertical dimension were encountered on top of this layer. Cobbles and boulders up to 0.4m were also encountered on top of the bedrock in BH-16S. Sand and gravel were also encountered overlying the bedrock in BH-15S and in some of Peto's boreholes drilled close to the northern end of the approach embankment (BH 612-31N).

## **5.8 Bedrock**

Bedrock of the Canadian Shield was encountered underlying the soils at the approximate elevations shown below.



**Table 5.3 – Bedrock Elevation**

<b>SBL BRIDGE</b>		
<b>Foundation Element</b>		<b>Bedrock Elevation (m)</b>
South Abutment	BH-1S	222.6
	BH-2S	221.6
	BH-3S	224.3
	BH-4S	225.4
	BH-5S	225.2
Pier 1S	BH-6S	216.0
	BH-10S	212.6
Pier 2S	BH-11S	201.4
	BH-12S	203.5
Pier 3S	BH-13S	181.3
	BH14-S	182.4
North Abutment	BH-15S	171.9
	BH-16S	172.4
<b>NBL Bridge</b>		
South Abutment	BH-1N	224.1
	BH-2N	223.3
	BH-3N	223.2
	BH-4N	223.5
	BH-5N	223.2
Pier 1N	BH-6N	208.4
	BH-7N	209.3
Pier 2N	BH-8N	196.5
	BH-9N	198.0
Pier 3N	BH-10N	179.1
	BH-11N	179.0
North Abutment	BH-12N	176.2
	BH-13N	177.9

Table 5.3 shows that the bedrock elevation at the bridge site varies from EL.225.4m at the South Abutment of the SBL Bridge to EL.171.9 at the North Abutment of the same bridge. Bedrock is also exposed at the northern portion of the approach embankment north of the bridges.

The bedrock at the site consists of Precambrian rock, consisting of granite and bands of diorite of the Central Gneiss Belt. The granite is grey in colour, with dark grey, green and red sub-horizontal bedding and high quartz content. The rock was fresh to medium weathered, strong to extremely strong with Unconfined Compression Strength (UCS) values inferred from Point Load Tests ranging from 50 to 330MPa. Details of the Point Load Test results are presented in Figures B27 and B28, Appendix B, and are summarized in Table 5.4. Discontinuities in the cores were slightly rough and were sub-vertical and sub-horizontal with spacing typically described as moderate to wide. Rubble zones were noticed in some of the cores ranging from 0.1m to 0.8m in thickness. With the exception

of clay filled discontinuities encountered approximately 2m below the top of bedrock in BH-6N, at the location of Pier 1N, the discontinuities were tight and clean.

Surface mapping of the rock outcrop at the locations of the south abutment footings identified three main sets of nearly orthogonal discontinuities with the following characteristics:

**Table 5.5 – Bedrock Structural Characteristics**

Joint Set #	Dip Direction	Dip
1 (Bedding)	140°	40°
2	320°	37°
3	50°	52°

The dip direction and dip angles shown above were measured in a relatively small area at the footing locations and do not necessarily represent the conditions elsewhere at the site or the characteristics of secondary joint systems.

Large rock blocks, with typical maximum dimension of 2m, were present at the surface at the toe of the rock outcrop and the south bank of the swamp. These blocks have detached from the parent rock and have toppled or slid along the edge of a wedge formed by joint sets #2 and #3 above.

## **5.9 Water Levels**

Attached Table 5.6 presents a summary of the groundwater levels observed in the standpipe piezometers installed as part of the program described herein as well as some of the wells installed by Peto during their 2003 investigation. It should be noted that all standpipe piezometers installed by Thurber were sealed at the interface between the bedrock and overlying soil. Peto's piezometers were sealed in the overburden soils, at relatively shallow depths.

The groundwater levels in Peto's piezometers indicate that since 1993 the groundwater level in the overburden soil raised between 1.3m to 4.3m between April 2003 and June 2004, to depths ranging from 2.2m to 6.5m. These groundwater levels are consistent with the groundwater levels inferred from piezocone tests, summarized in Appendix PPD of ConeTec's data report, included in Appendix D.

The piezometric level in the swamp, measured at the interface of the bedrock and the overlying soil was at surface, at approximate EL.219. North of the railway property area, the piezometric level ranged from EL.218.2 to EL.220.4. One standpipe piezometers (BH-13S) installed at the west end of proposed Pier 3S, at the toe of the slope between the

low-lying area and the uplands north of it showed artesian head, 1.12m above ground surface.

It should be noted that the groundwater levels discussed above are relatively short term and may not be stabilized. In addition, seasonal fluctuations in the groundwater table are also anticipated.

Engineering analysis and report prepared by:



Paulo Branco, Ph.D., P.Eng

Project Engineer, Principal



Report Reviewed by:

P. K. Chatterji, Ph.D., P.Eng.

Review Principal

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Geocres Number: 41I-184**

**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**6 INTRODUCTION**

This part of the report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to select and design a suitable foundation system and approach fills for the proposed structures.

The design will include the construction of two overhead structures over the swamp and approach embankments up to 6.3m high and up to 280m long north of the bridges. The two proposed overhead structures will be parallel to each other and are expected to consist of four spans (SBL: 32.5m/40.0m/40.0m/32.5m; NBL: 37.5m/40.0m/40.0m/35.0m) with a total of five foundation elements, two abutments and three piers for each structure. The approaches to the bridge will be founded on overburden soils that include thick compressible silty clay soils.

The stability and time-dependent settlements and the impact on the construction schedule and the long-term performance of the proposed structures and approaches are also analysed in this section of the report.

The discussion and recommendations presented in this report are based on our understanding of the project and on the factual data obtained in the course of this investigation, together with the factual data from a previous investigation by Peto MacCallum Ltd. (Peto).

**7 STRUCTURE FOUNDATIONS**

**7.1 Foundation Alternatives**

The subsurface conditions at the foundation elements are summarized in Table 7.1 below:

**Table 7.1 – Compressible Soil Thickness and Bedrock Elevation**

Foundation Element	Borehole	Thickness of Compressible Cohesive Soils (m)	Bedrock Elevation (m)	Bedrock Depth Below Ground Surface (m)
SBL Bridge				
South Abutment	BH-1S	0	222.6	0.4
	BH-2S	0	221.6	0.1
	BH-3S	0	224.3	0.0
	BH-4S	0	225.4	0.1
	BH-5S	0	225.2	0.0
Pier 1S	BH-6S	0	216.0	3.0
	BH-10S	2.9	212.6	6.6
Pier 2S	BH-11S	12.3	201.4	17.7
	BH-12S	10.3	203.5	15.8
Pier 3S	BH-13S	29	181.3	37.9
	BH-14S	30.5	182.4	36.7
North Abutment	BH-15S	29.7	171.9	51.5
	BH-16S	33.5	172.4	51
NBL Bridge				
South Abutment	BH-1N	0	224.1	0.0
	BH-2N	0	223.3	0.1
	BH-3N	0	223.2	0.1
	BH-4N	0	223.5	0.1
	BH-5N	0	223.2	0.1
Pier 1N	BH-6N	4.8	208.4	10.6
	BH-7N	6.7	209.3	10.0
Pier 2N	BH-8N	16.3	196.5	22.8
	BH-9N	15.7	198.0	21.1
Pier 3N	BH-10N	29.5	179.1	41.2
	BH-11N	27.2	179.0	42.0
North Abutment	BH-12N	29.6	176.2	48.0
	BH-13N	26.9	177.9	46.1

Five foundation types have been considered:

- Spread footings on native soil
- Spread footings on bedrock
- Spread footings on engineered fill
- Drilled shafts (also referred to as caissons or bored piles)
- Driven piles

These foundations alternatives are discussed below.

## **7.2 Spread Footings on Native Soil**

Spread footings bearing on the native soils are not considered to be suitable for the support of the foundations at this structure on account of the low values of geotechnical resistance that would be available and on account of the potential for large immediate and long-term settlements.

## **7.3 Spread Footings on Bedrock**

### **7.3.1 General**

The bedrock is at surface or shallow at the south abutments and Pier 1S. At the remaining foundation element locations the bedrock is deep and spread footings founded on bedrock are not feasible.

### **7.3.2 SBL and NBL South Abutments**

A spread footing founded on sound bedrock is considered feasible at the south abutment of both bridges, where the bedrock is either exposed at ground surface or covered by shallow organic soil.

The top surface of the bedrock should be stripped of all overburden and be cleaned. All shattered and loosened rock fragments should be removed from the footprint of the footing and replaced with mass concrete fill. The footing may be designed on the basis of a concentric, vertical geotechnical resistance of 5,000kPa at factored ULS. The SLS condition will not govern for footings founded on bedrock. The above resistance value is for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clauses 6.7.3 and 6.7.4.

The established bedrock elevations at the south abutments shown above indicate that there is a difference in elevation of the top of bedrock of 3.8m and 0.9m across the South Abutments of the SBL and NBL Bridges, respectively. Mass concrete fill underneath the footing should be used to compensate for difference in elevations of the top of bedrock.

The Contractor should scale all loosened rock from the rock face in front of the footing and the Contract Administrator should retain a rock slope stability expert to examine the cut. Where the rock face below the foundation elements develops potentially unstable wedges or where over-break occurs, the Contractor should place mass concrete fill or install rock bolts as required. The remedial work should be designed by and carried out under the direction of the rock slope stability expert hired by the Contract Administrator. The contract should include an NSSP to this effect.

### **7.3.3 Pier 1S**

The bedrock at this location dips to the northeast with a difference in elevation of 3.4m between the southwest and the northeast corners of the footing. The bedrock is overlain by

water bearing soils and peat and the groundwater table is at surface. This option will require a watertight excavation to the top of bedrock. The shoring system should consist of sheet piles around the perimeter of the excavation to achieve a sufficiently dry base on which to construct the footing and it should be advanced to expose the top of bedrock. The potential for the presence of cobbles and boulders in the soils overlying the bedrock, which will make sheet pile installation difficult, the sloping bedrock surface and the fact that it will be difficult to control groundwater seepage through discontinuities in the bedrock make this alternative impractical.

#### 7.3.4 Lateral Resistance on Bedrock

Initial calculations of the horizontal resistance may be carried out using a value of 0.7 for the ultimate friction factor of concrete poured on rock.

If the frictional component is insufficient, the horizontal resistance may be increased by dowelling into the rock mass. Dowels are considered to be comparatively short steel bars that may be assumed to provide only shear resistance. If vertical resistance in tension is required, rock anchors should be included in the design.

The dowel is treated as a fully embedded pile in the rock and hence will fail when the ultimate lateral resistance of the rock or grout is exceeded. Using a lower bound value of 20MPa for the strength of the rock or grout, an ultimate horizontal resistance of 1.5MN may be assumed for a 50 mm steel dowel embedded 500 mm into the rock. The depth of embedment is measured below the bearing surface prepared to receive the concrete footing.

The shearing resistance of the dowel must be checked structurally.

#### 7.4 Spread Footings on Engineered Fill

The use of spread footings bearing on engineered fill pads is considered to be feasible at the SBL and NBL south abutments provided all overburden is removed and the engineered fill is constructed on the bedrock. This alternative is not recommended for Pier 1S due to the reasons outlined in Section 7.3 above.

After the exposure and cleaning of the bedrock surface, the top of bedrock surface may slope unfavourably for stability against sliding at the interface of the bedrock and engineered fill. Bedrock surfaces steeper than 20° with horizontal and dipping towards the swamp should be benched to provide for horizontal surfaces at the interface of the engineered fill and underlying bedrock surface.

The Contractor should scale all loosened rock from the rock face in front of the footing and the Contract Administrator should retain a rock slope stability expert to examine the cut. Where the rock face below the abutments develops potentially unstable wedges or where over-break occurs, the Contractor should place mass concrete fill or install rock bolts as required. The remedial work should be designed by and carried out under the direction of the rock slope stability expert hired by the Contract Administrator. The contract should include an NSSP to this effect.

Given the soils shown to exist at the site, footings on engineered fill pads bearing on the native soil are not recommended due to the possibility of unacceptably large settlements.

If a footing on engineered fill bearing on bedrock is used, it may be designed on the basis of the following concentric, vertical geotechnical resistances:

- 900kPa at factored ULS
- 350kPa at SLS

The engineered fill must be founded on the bedrock at the elevations given in the table provided in Section 7.1 and a minimum thickness of 2.0 m of engineered fill must be maintained between the underside of the concrete and the top of the bedrock.

The engineered fill must consist of OPSS Granular "A" placed in 150 mm lifts and compacted to 100% of its SPMDD at  $\pm 2\%$  of optimum moisture content and generally conforming to the geometry illustrated in Figure 7.1.

The resistance values above are for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

For footings designed on the basis of the geotechnical resistance values given above, total settlement under a footing is not expected to exceed 25 mm. Differential settlements are not expected to exceed 15 mm across the width of the structure.

The sliding resistance of mass concrete poured on a compacted Granular "A" pad over bedrock and between the engineered fill and bedrock may be computed on the basis of ultimate friction factors of 0.7 and 0.5, respectively.

## **7.5 Drilled Shafts (Caissons or Bored Piles)**

The foundations may also be supported on drilled shafts founded in the bedrock.

Due to the relatively high groundwater table and presence of water bearing cohesionless soils immediately above the bedrock, it will be necessary to advance a temporary liner into the top of the bedrock to exclude soil from the drilled shaft. In addition, a qualified geotechnical engineer or technician should visually inspect the top of bedrock to assess the bedrock quality and resistance. Hence, a temporary liner and dewatering will be required for quality control.

The axial capacity of drilled shafts will be derived from shaft and base resistance in the bedrock. Drilled shafts may be designed on the basis of a concentric, vertical geotechnical resistance of 5,000kPa for end bearing and 1,250kPa for shaft friction in bedrock at factored ULS. The SLS condition will not govern on the bedrock. The caisson should be founded on fresh, intact or tightly jointed bedrock, to a minimum depth of 1.5m below the bedrock surface.

The caissons must be installed in accordance with SP 903S01.



As it is not considered practical to install caissons on a batter, lateral loads must be resisted by earth pressure as described in Section 7.12 below or by socketing the caissons into bedrock.

## 7.6 Driven Piles

The stratigraphy encountered at this site is considered to be suitable for the use of steel piles driven to bedrock to support the foundations. The use of H-section piles is recommended and the following concentric, vertical, geotechnical resistances (factored ULS) will be available for the pile sections shown in Table 7.2 below.

**Table 7.2 – Pile Resistances**

Pile Section	ULS <sub>f</sub>
HP 310 X 110	2,000 kN
HP 310 X 132	2,400 kN
HP 310 X 152	2,750 kN
HP 360 X 132	2,400 kN

The SLS case will not govern for piles driven to bedrock.

The estimated pile tip elevations, based on the bedrock elevations, are shown in Table 7.1.

The stratigraphy encountered at SBL and NBL South Abutments consists of shallow bedrock. The proposed Highway 69 SBL grade lies 8.2m to 12.0m above the top of bedrock and the underside of an abutment stem is anticipated to lie approximately 2.5m to 6.5m above the bedrock. The proposed Highway 69 NBL grade lies 9.7m to 10.6m above the top of bedrock and the underside of an abutment stem is anticipated to lie approximately 4m to 5m above the top of bedrock. In its existing condition, the site at the South Abutment of both bridges is not considered suitable for driven piles and consequently not suitable for integral abutment. However, it is recognized that integral abutment bridges, which require pile foundations, offer significant long term advantages. Therefore the installation procedures outlined below are recommended for the use of integral abutments at the bridge south abutments.

The recommended minimum pile length below the abutment is 5 m, consisting of 3 m in loose sand and a minimum of 2 m driven into resisting material below. An integral abutment structure may be designed at this site if the foundation area is prepared as follows:

- Excavate bedrock to form a trench 2 m wide, centred on the bridge bearings and extending 1 m beyond the edge of the structure to either side. The base of the trench must be at least 5 m below the underside of the abutment.
- Backfill the trench and complete construction of engineered fill up to a level 3 m below the underside of the abutment using OPSS Granular "A" compacted in accordance with OPSS 501.

- Proceed with normal integral abutment construction, driving the piles through the granular backfill to seat on bedrock.

Relatively short piles are also anticipated at Pier 1S, where the bedrock surface lies between 3.0 and 6.6m below ground surface. If short piles cannot provide sufficient resistance to horizontal loading, it is recommended that the piles be socketed into rock for a distance of 1m with the rock socket backfilled with concrete. Construction of sockets will require mobilization of a separate rig and will add significantly to the cost of the foundation.

## 7.7 Recommended Foundation System

A comparison of foundation alternatives based on advantages and disadvantages of each foundation alternative is included in Table 7.3.

Spread footings and steel piles are feasible for the south abutment of both bridges. It is understood, however, that the use of integral abutments is preferred for this site which requires piled foundation socketed into bedrock.

The use of a spread footing founded on bedrock for Pier 1S is not considered practical due to the relatively deep and sloping bedrock condition in association with the difficulty in maintaining the base of excavation relatively dry during construction. Therefore steel pile driven and possibly socketed in bedrock is recommended for this foundation element.

For all other foundation elements, based on the local experience and the difficulty in inspecting the base of drilled shafts seated on bedrock, the recommended foundation system is steel H-piles driven to bedrock. The design of the north approach embankments should not include the use of rock fill at the north abutment locations to allow for pile installation. In view of the large depths of pile installation, it is recommended that the larger pile sections be used in this project.

The following sections provide additional recommendations for driven piles. Additional design recommendations for drilled shafts may be provided if this foundation type is selected for this site.

## 7.8 Downdrag

Depending on the design alternative selected for the north embankments, long-term settlements of the foundation soils will result in downdrag forces on the north abutment piles. Estimates of downdrag forces per pile are summarized in Table 7.4. Table 7.4 has been prepared assuming two conditions:

- Downdrag forces will be mobilized above EL.185, i.e. above the silty clay to silt layer. This condition will prevail for most design alternatives for the approach embankments discussed later this report.

- Downdrag forces will be mobilized above EL.200, i.e. above the CI-CH silty clay sub-layer. This condition will prevail if the embankment is constructed using Rigid Expanded Polystyrene (REP or EPS) fill as discussed latter in this report.

**Table 7.4 – Downdrag Forces on North Abutment Piles**  
**SBL and NBL Bridges**

<b>Neutral Plane at approximate EL.185 (above Clayey Silt to Silt layer)</b>				
Pile Type	HP 310x110	HP 310x132	HP 310x152	HP 360x132
Estimated downdrag force (*)	4884 kN	4955 kN	5058 kN	5722 kN
Factored downdrag force (f = 1.25)	6105 kN	6194 kN	6323 kN	7153 kN
<b>Neutral Plane at approximate EL.200 (above Silty Clay – CI-CH sub-layer)</b>				
Pile Type	HP 310x110	HP 310x132	HP 310x152	HP 360x132
Estimated downdrag force (*)	2263 kN	2296 kN	2344 kN	2651 kN
Factored downdrag force (f = 1.25)	2829 kN	2870 kN	2930 kN	3314 kN

(\*) Downdrag forces have been calculated assuming that the negative skin friction will be mobilized at the outside perimeter of the “H” pile, between the underside of the pile cap and the Neutral Plane. The analysis was carried out assuming the subsurface conditions and drained shear strength values calculated based on “Beta” values shown in Table 10.1 and Table 10.2.

Table 7.4 shows that the downdrag forces are high and, depending on the results of the analysis of the structural capacity of the piles at the Neutral Plane carried out by others, measures to reduce negative skin friction such as the use of bitumen coating or an outside casing surrounding the pile may be required. Downdrag loads could be significantly reduced by sub-excavating the native soils to 1.5m depth within the embankment footprint and within 40m of the abutment and replacing it with EPS above that depth to the underside of the pavement structure.

### **7.9 Pile Tips**

The H-piles for the recommended foundation scheme will be driven to bedrock and will encounter cobbles and boulders that overlie the bedrock. In addition, due to sloping bedrock surface at this site, it is recommended that the pile tips be fitted with rock points. A suitable rock point is the Titus Steel Company Rock Injector, APF Hard Bite, or equal from another approved manufacturer.

### **7.10 Pile Installation**

Pile installation should be in accordance with Special Provision No. 903S01.

### **7.11 Pile Driving**

The appropriate note for the foundation drawing is Note 6, i.e. “Piles to be fitted with Rock Points and driven into bedrock in accordance with OPSS 903”.

## 7.12 Lateral Resistance of Piles

Horizontal loads may be resisted either by batter piles or through the lateral resistance developed by the pile acting against the surrounding soil.

The lateral resistance of the piles must be calculated based on the horizontal coefficients of subgrade reaction ( $k_s$ ) and taking account of the ultimate lateral resistance ( $p_{ult}$ ) provided below:

- Granular engineered fill (below the 600mm CSP for integral abutments):

- $k_s = f \cdot z / D$  (MN/m<sup>3</sup>)

- Where  $f = 18 \text{ MN/m}^3$

- $z$  = depth below ground surface

- $D$  = pile diameter or width in a direction perpendicular to the pile movement

- $p_{ult} = \gamma \cdot z^3 \cdot K_p$  (kPa)

- Where  $\gamma = 22 \text{ kN/m}^2$

- $z$  = depth below ground surface (m)

- $K_p = 3.7$  (passive earth pressure coefficient)

- Peat

- $k_s = 0$  (neglect lateral resistance of peat)

- Sand /Silt (Above EL.212):

- $k_s = f \cdot z / D$  (MN/m<sup>3</sup>)

- Where  $f = 2 \text{ MN/m}^3$

- $z$  = depth below ground surface

- $D$  = pile diameter or width in a direction perpendicular to the pile movement

- $p_{ult} = \gamma \cdot z^3 \cdot K_p$  (kPa)

- Where  $\gamma = 19 \text{ kN/m}^2$

- $z$  = depth below ground surface (m)

- $K_p = 3.3$  (passive earth pressure coefficient)

- Silty Clay CL (Between EL.212 and EL.200 at the north abutments):

- $k_s = 6.5 / D$  (MN/m<sup>3</sup>)

- Where  $D$  = pile diameter or width in a direction perpendicular to the pile movement (m)

$$p_{ult} = 400 \text{ kPa } (\cong 9 * C_u)$$

- Silty Clay CI-CH (Between EL.200 and EL.185 at the north abutments):

- $k_s = 13/D \text{ (MN/m}^3\text{)}$

Where  $D$  = pile diameter or width in a direction perpendicular to the pile movement (m)

$$p_{ult} = 800 \text{ kPa } (\cong 9 * C_u)$$

- Clayey Silt to Silt (Above EL.172):

- $k_s = f * z/D \text{ (MN/m}^3\text{)}$

Where  $f = 2 \text{ MN/m}^3$

$z$  = depth below ground surface

$D$  = pile diameter or width in a direction perpendicular to the pile movement

- $p_{ult} = \gamma * z * 3 * K_p \text{ (kPa)}$

Where  $\gamma = 19 \text{ kN/m}^2$

$z$  = depth below ground surface (m)

$K_p = 3.0$  (passive earth pressure coefficient)

- Bedrock (for HP piles grouted into a 0.4m diameter hole cored into bedrock):

- $k_s = 680 \text{ MN/m}^3$  at bedrock surface to  $3,400 \text{ MN/m}^3$  at 1.2m below the top of bedrock;  $3,400 \text{ MN/m}^3$  below 1.2m depth

$p_{ult} = 120 \text{ MPa}$  at bedrock surface to  $610 \text{ MPa}$  at 1.2m below the top of bedrock;  $610 \text{ MPa}$  below 1.2m depth

Spring constant ( $K_s$ ) and ultimate spring load ( $P_{ult}$ ) values for numerical analysis of the integral abutment piles can be obtained by multiplying the  $k_s$  and  $p_{ult}$  values above, respectively, by the pile diameter or width (in a direction perpendicular to the pile movement) and the vertical distance between nodal points of the numerical model mesh along the pile.

### 7.13 Frost Protection

The depth of earth cover required to provide frost protection for footings and pile caps at this site is 2 m.

It is recommended that the full depth of soil cover be provided unless adequate synthetic insulation is incorporated in the design to replace the soil cover. It is recommended that the frost protection not be reduced at this site, due to the nature of the native soils and the high groundwater table.

It should be noted that rock fill does not provide the insulation value of soil cover. Where rock fill is used as backfill or for the construction of forward slopes in frost sensitive locations, consideration should be given to incorporating synthetic insulation.

Rigid, extruded polystyrene (EPS) insulation may be used and it may be assumed that 25 mm of this insulation provide protection equivalent to 600 mm of soil cover.

#### **7.14 Abutment Considerations**

##### **7.14.1 Retained Soil Systems**

Retained Soil System walls are not considered suitable for the North Abutment due to the potential for large settlement during construction and long-term settlements due to secondary consolidation.

Retained soil system (RSS) walls may be used at the South Abutment provided that the levelling pad for the RSS wall is formed directly on the exposed bedrock, mass concrete fill or on a pad of engineered fill seating on bedrock. Engineered fill should be designed in the same manner as the engineered fill to support foundations as described elsewhere in this report. The geotechnical resistance of the bedrock or engineered fill is as stated elsewhere in this report.

RSS walls should be specified to be “High Performance” and “High Appearance”.

The contract drawings should include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall.

The global stability of an RSS wall founded as described above will be satisfactory.

The internal stability of the RSS should be analysed by the supplier/designer of the proprietary product selected for this site.

The settlement of a wall founded on engineered fill pad placed on bedrock is expected to be small and should occur essentially as the RSS is constructed.

##### **7.14.2 Integral and Conventional Abutments**

Integral and conventional abutments are considered suitable for this site. To provide the required flexibility in the piles of integral abutments, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP filled with sand in accordance with standard integral abutment design procedures.

#### **7.15 Backfill to Abutments**

##### **7.15.1 EPS Fill Approach Embankments**

If EPS is selected as the fill material for the approach embankment, a geosynthetic sheet drain should be placed along the back of the abutment wall, at the interface of the EPS and abutment wall.

The design of the abutment should incorporate a subdrain at the base of the granular backfill, as applicable.

#### 7.15.2 Conventional Approach Embankments (Rock or granular fill)

In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted. The rock fill used as backfill to the abutment should be limited to fragments no greater than 300 mm and including adequate spalls to fill voids in the rock fill.

In the case of integral abutments, the backfill should consist of granular material.

In all cases where the approach embankment consists of rock fill and the abutment wall is backfilled with granular fill, the granular backfill should consist of OPSS Granular "B" Type II.

The backfill to the abutment walls should be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill should be placed to the extents shown in OPSD 3501.000, and rock backfill should be placed to the extents shown in OPSD 3505.000.

Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with SSP 105S10.

The design of the abutment should incorporate a subdrain as shown in OPSD 3501.000 or OPSD 3505.000, as applicable.

## 8 EXCAVATION

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils at this site and any earth fill are classed as follows:

- Soils located in the low lying area (Ground surface at approximate EL.219): Type 4 Unwatering will be required prior to excavation below the existing ground surface.
- Soils located in the uplands, north of the low lying area: Type 3 soils. Unwatering will be required prior to excavation below the groundwater level.

## **9 SOUTH APPROACH EMBANKMENTS**

### **9.1 General**

The south approach embankments to the proposed bridges will be founded mainly on bedrock that is either benched or sloped unfavourably towards the swamp. The toes of the forward slopes encroach into the swamp where the subsurface conditions consist of shallow peat underlain by loose to firm fine sediments consisting of silt and silty and sandy clay to variable depth.

The main geotechnical issues associated with these embankments are the stability of the forward slopes during construction and long term settlement and lateral spread of the forward slopes.

### **9.2 Stability Analysis**

Stability analyses were carried out for the south approach embankments using the Bishop Modified method and G-Slope, developed by Mitre Software. It has been assumed that the forward slope will be constructed using blast rock sloped at 1.25H:1V, with the geometry and material properties shown on Figure C17 and Figure C19. In addition, it has been assumed that the loose and soft soils overlying the bedrock will be removed and replaced with blast rock.

The results of the stability analysis are shown on Figures C17 through C19 for abutments founded on piles (Figure C17 and C19) and for abutments founded on engineered fill (Figures C18 and C20). The stability results show that the factors of safety against sliding at the interface of the bedrock and overlying embankment ranges from 1.5 to 1.8 for all cases analysed, which are satisfactory.

### **9.3 Overburden Removal**

The bedrock surface should be stripped of all overburden and be cleaned within the footprint of the embankment. This includes the area within the swamp, where this operation will be carried out to depths up to 7m below the groundwater table. The excavation within the swamp should be carried out following the recommendations provided in Section 8 of this report. Stockpiled material, access roads and equipment should be kept at distances larger than the depth of the excavation measured from the crest of the slope. It is critical that overburden materials be completely removed from within the footprint of the embankment as shown on Figures C17 through C20. The backfill material in the swamp should consist of rock fill sloped at 1.25H:1V.



## 10 NORTH APPROACH EMBANKMENTS

### 10.1 General

The north approach embankments are up to 6.3m high to top of pavement and will be founded on foundation soils that include thick, compressible deposits of silty clay. The geotechnical issues associated with the north approach embankments are:

- Stability during construction and during a seismic event
- Large long-term settlements due to primary and secondary consolidation
- Large post-construction lateral deflections of the foundation soils in the area of the north abutment piles associated with settlements due to secondary consolidation

Embankment configurations and construction sequences selected for analysis at this site are shown in Table 10.3. The embankment configurations shown in Table 10.3 include different materials, surcharge and preloading alternatives. It is understood that large rock cuts north and south of the site will provide enough blast rock for construction of the embankments. Notwithstanding this fact, two of the design alternatives, C and D include the construction using sand fill in order to allow for installation of piles and/or removal of the sand fill for replacement with EPS fill. Table 10.3 also includes the alternative where the embankment is constructed to its final shape using EPS fill protected with 1m of sand. This alternative does not include preloading or sub-excavation of the foundation soils.

Engineering analyses were carried out for the different embankment configuration in order to address the geotechnical issues outlined above as follows:

- Selection of cross sections for analysis that represent typical subsurface conditions and embankment configurations with respect to embankment height and width.
- Stability analysis to identify the need for stabilizing berms and construction staging;
- Stability and lateral spread analysis of the approach embankments subject to seismic loading;
- Settlement analysis to identify the need for and the spacing of wick drains to accelerate settlement and to accommodate the construction schedule;
- Analysis of post-construction settlements and lateral deflections to assess minimum surcharge requirements and the long term performance of the approach embankments and abutment piles.

These aspects of the design are addressed in the following sections.

### 10.2 Subsurface Models

The following sections along the north approach embankment have been selected for analysis based on their relatively uniform subsurface conditions and embankment geometry.

#### SBL Approach Embankment:

North Abutment	– St.10+550 to St.10+600 (0 to 50m from the abutment)
North Approach 1	– St.10+600 to 10+675 (50 to 125m from the abutment)
North Approach 2	– St.10+675 to Dill TWP St.10+010 (125 to 220m the abutment)
North Approach 3	– Dill TWP St.10+010 to 10+040 (220 to 250m from the abutment)

#### NBL Approach Embankment:

North Abutment	– St.10+580 to St. 10+620 (0 to 40m from the abutment)
North Approach 1	– St.10+620 to 10+675 (40 to 95m from the abutment)
North Approach 2	– St. 10+675 to Dill TWP St. 10+010 (95 to 190m from the abutment)
North Approach 3	– Dill TWP St.10+010 to 10+040 (from 190 to 220m from the abutment)

Figures 10.1 through 10.6 present a summary of relevant soil properties in sections North Abutment, North Approach 1 and North Approach 2 listed above for both SBL and NBL bridges. Tables 10.1 and 10.2 present the simplified subsurface models for each of the sections selected for analysis.

### 10.3 Stability Analysis

#### 10.3.1 General

Typical simplified stratigraphic profiles were selected for the subsurface conditions and embankment geometries in order to analyse the embankment stability. Tables 10.1 and 10.2 present simplified subsurface conditions for the SBL and NBL approach embankments, respectively. Within each of the embankment sections listed in the first column of Tables 10.1 and 10.2, the maximum embankment height was selected for analysis.

The stability analysis was carried out based on the following assumptions:

- Factor of Safety Requirements:
  - Short Term:  $FS_{min} = 1.3$
  - Long Term:  $FS_{min} = 1.5$
- Embankment materials and slopes:

Two types of embankment materials were analyzed:

- Select Subgrade Material (SSM): 2H:1V for permanent slopes and 1.5H:1V for temporary slopes
- Rock Fill: 1.25H:1V side slope

It was assumed that the body of the embankment will be constructed using rock fill and that the surcharge and the temporary forward slopes will be constructed using SSM.

- Surcharge:
  - A surcharge of 3m above the top of pavement was assumed in the analysis. As it will be shown later in this report, surcharges lower than 3m would not result in sufficient surcharging of the foundation silty clay deposit. At the abutment locations it has been assumed that the surcharge will extend to beyond the centreline of the bearings towards the bridge.
- Staging:
  - It was assumed that the embankment construction will be carried out in one or two stages as required.
- Site Preparation:
  - All organic soils will be removed within the footprint of the embankment and side berms
- Limit Equilibrium Analysis:
  - Bishop Modified using G-Slope, developed by Mitre Software
- Soil Shear Strength:
  - Undrained shear strength ( $S_u$ ) for cohesive soils as shown in Table 10.1 and Table 10.2. For vertical effective stresses ( $\sigma'_v$ ) larger than the pre-consolidation pressure ( $p'$ ),  $S_u$  was assumed equal to  $0.21 * \sigma'_v$  for the Silty Clay – CH deposit and  $0.25 * \sigma'_v$  for the other cohesive deposits.
  - Drained shear strength ( $\phi'$ ) for cohesionless soils as shown in Table 10.1 and Table 10.2.
- Pore pressure generation and dissipation:
  - Generation of excess pore pressures (EPP) upon undrained loading of the compressible and cohesive deposits is calculated assuming a  $B_{bar}$  (ratio of EPP over vertical total stress) of 0.9
  - Dissipation of EPP between loading stages was assumed to be equal to 98%
- Groundwater Table:
  - 1m to 2m above the swamp elevation.

The results of the stability analysis are presented in Figures C1 through C9 (SBL embankment) and Figures C10 through C16 (NBL embankment), Appendix C. An analysis of these figures indicates the following:

- SBL Approach:
  - Temporary Forward Slope (Figures C1, C2 and C3): The temporary forward slope (1.5H:1V) should be constructed with SSM in two stages with 98% dissipation of excess pore pressures (EPP) between stages. Alternatively, for construction in one stage a benched slope (11.5m wide berm, 5.8m above the swamp elevation), as shown on Figure C3, will be required
  - Abutment – Final Configuration (Figure C4 and C5): The proposed abutment configuration is stable in the short (undrained) and long term (drained).
  - Embankment Side Slopes (Figures C6 through C9): The approach embankment is stable under short (undrained) and long term (drained) conditions for embankments consisting of rock fill, sand and a combination of both
- NBL Approach:
  - Temporary Forward Slope (Figure C10): The temporary forward slope can be constructed using SSM in one stage with a forward slope at 1.5H:1V
  - Abutment – Final Configuration (Figure C11 and C12): The proposed abutment configuration is stable in the short (undrained) and long term (drained).
  - Embankment Side Slopes (Figures C13 through C16): The approach embankment is stable under short (undrained) and long term (drained) conditions for embankments consisting of rock fill, sand and a combination of both

## 10.4 Settlement Analysis

### 10.4.1 General

The settlement analysis was carried out assuming subsurface models shown in Tables 10.1 and 10.2 for the embankment configurations shown in Table 10.3.

Settlement analyses were carried out as follows:

- One-dimensional primary consolidation analysis for initial embankment (including surcharge if required) configurations A, C, D and E: no wick drains
- Pseudo three-dimensional consolidation analysis for initial embankment configurations A, C, D and E: with wick drains
- One-dimensional secondary consolidation analysis for final configurations (after removal of surcharge and construction of pavement) B, D and E

### 10.4.2 One-Dimensional Consolidation Analysis: No wick drains

One-dimensional consolidation analyses were carried out in order to:

- Assess the total settlement (time-independent) due to primary consolidation
- Analyse the dissipation of pore pressures with time and establish the need for wick drains;
- Provide input for the vertical consolidation component in the wick drain design

The analysis was carried out using the finite difference software Consol Version 2.0, developed at Virginia Polytechnic Institute and State University. The program allows the one dimensional consolidation analysis of multi-layered soil masses, taking into account non-linear constitutive law, variable parameters as a function of the over-consolidation ratio, impeded drainage and variable boundary conditions.

The analyses were carried out assuming that the embankment construction will occur in one stage, instantly at the start of construction. This is a simplified model of the actual construction process in which several days or weeks are typically required to complete each of the construction stages. This approach over predicts excess pore pressures since it assumes that there will be no dissipation of pore pressures during fill placement.

The results of the one-dimensional primary consolidation settlement analysis are presented in Tables F1 through F8 and Figures F1 through F8, Appendix F and summarized in Tables 10.4 and 10.5. The settlements calculated based on the assumption that the silty clay is normally consolidated are considered upper bound values. The settlements calculated based on the assumption that the silty clay is slightly over-consolidated, with the over-consolidation ratios shown in Tables 10.1 and 10.2 are considered most likely values. The analysis of settlements in the remainder of this report focuses on the most likely settlement values. The analysis of Tables 10.4 and 10.5 indicate the following:

- Settlements due to primary consolidation are up to 0.7m behind the abutments up to 0.8m elsewhere
- The time required for 98% completion of primary consolidation is more than 2 to 4 years for the north approach embankments located within 250m and 220m north of the SBL and NBL north abutments, respectively. This time is reduced to 1 to 2 years for distances greater than 250m to 220m north of the SBL and NBL north abutments, respectively.
- The minimum time required ( $t_{min}$ ) for removal of surcharge to stabilize settlements due to primary consolidation varies depending on the abutment configurations before and after removal of surcharge as shown below:
  - Configuration A to B (Rock fill embankment with 3m surcharge):
    - SBL
      - $t_{min}$  = 1 to 2 years within 50m of the North Abutment
      - $t_{min}$  = less than 1 year beyond 50m of the North Abutment

#### NBL

- $t_{\min} = 2$  to 3 years within 40m of the North Abutment
- $t_{\min} = 1$  to 2 years between 40m and 95m north of the North Abutment
- $t_{\min} =$  less than 1 year beyond 95m of the North Abutment

- Configuration C to D (Sand fill embankment with 3m surcharge):

#### SBL

- $t_{\min} = 3$  to 4 years within 50m of the North Abutment
- $t_{\min} = 1$  to 2 years between 50m and 220m of the North Abutment
- $t_{\min} =$  less than 1 year beyond 220m of the North Abutment

#### NBL

- $t_{\min} = 3$  to 4 years within 40m of the North Abutment
- $t_{\min} =$  less than 1 year beyond 95m of the North Abutment

- Configuration D to E (Sand fill temporary embankment replaced with EPS embankment):

#### SBL and NBL

- $t_{\min} =$  less than 1 year anywhere behind the North Abutment
- Configuration E to E (EPS embankment in its final configuration, without surcharge and without sub-excavation)
  - $t_{\min}$  minimum is not applicable since there is no surcharge in this case

Removal of the surcharge at  $t_{\min}$  would, in most cases considered herein, bring the silty clay to a normally consolidated state, a condition that is undesirable because it is associated with large post-construction settlements as discussed later in this report.

- The results of the settlement analysis above show that wick drains will be required to accelerate the dissipation of excess pore pressures in the silty clay deposit if the time available between the end of the embankment construction to the top of surcharge and the removal of surcharge is less than 2 to 4 years.

#### 10.4.3 Pseudo Three-Dimensional Consolidation Analysis: With Wick Drains

The method by Hansbo<sup>1</sup> (1960) was used for the wick drain design. The method includes well resistance and disturbance factors due to the wick drain installation. EPP dissipation

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<sup>1</sup> Hansbo, S. (1960). Consolidation of clay, with special reference to influence of vertical sand drains. Swedish Geotechnical Institute, Proceedings No.18 (1960)

due to vertical drainage was coupled with EPP dissipation due to horizontal drainage into the wick drains according to the following equation:

$$U = 1 - (1 - U_v) * (1 - U_h)$$

where U is the combined total percentage consolidation and  $U_v$  and  $U_h$  are the percentage consolidation values due to vertical and horizontal drainage, respectively, divided by 100. The design parameters used in the wick drain analysis and selected results of the analysis are summarized in Figures G1 and G2 in Appendix G. Since the wick drain design method described above does not allow inclusion of horizontal coefficient of consolidation values ( $C_h$ ) as a function of the pre-consolidation pressure ( $p'$ ) for the horizontal drainage portion of the analysis, the lowest value of  $C_h$  has been assumed for a specific test hole location. It has been assumed that the wick drains will be installed in a triangular pattern spaced at 1.5m, 1.75m and 2.0. The lateral extent of the wick drains will be 5m beyond the embankment toes in order to accelerate dissipation of excess pore pressures in the deeper portions of the compressible deposits that extend beyond the embankment footprint. It has been assumed that the wick drains will be terminated within the granular materials that underlie the silty clay and clayey silt deposits. Therefore the wick drain drainage length has been assumed equal to one half of the length of the wick drain.

Tables 10.4 and 10.5 show the time required for 98% of the settlements due to primary consolidation to occur for the embankment configurations prior to removal of surcharge, where applicable. Analysis of Tables 10.4 and 10.5 shows that construction windows ranging from 3 to 6 months would be required between the end of construction of the embankment to the top of surcharge, if required, and the surcharge removal.

#### 10.4.4 One-dimensional secondary consolidation analysis

Settlements due to secondary consolidation of the silty clay were calculated using the method by Mesri and Feng<sup>2</sup> (1991) based on the assumption that wick drains will be used and that settlements due to primary consolidation will be completed 6 months after the end of construction. The results of the analysis are presented in the tables and figures included in Appendix H which include settlements due to secondary consolidation 1, 3, 6, 10 and 20 years after the end of the pavement construction and are summarized in Tables 10.6 and 10.7. Although settlements due to secondary consolidation were carried out assuming that the silty clay deposit is both normally and over-consolidated, the following discussion only addresses the results of the analysis in which the clay was considered over-consolidated, which is believed to represent the most likely site conditions. The results of the analysis indicate the following:

- For all embankment configurations considered herein, with the exception of embankments located north of Station 10+010 in the Dill Township and between

<sup>2</sup> Mesri G and Feng T.W., 1991. "Surcharge to Reduce Secondary Consolidation" Geo-Coast '91, 3-6 Sept., 1991, Yokohama, pp.359-364

Stations 10+660 and 10+690, where the embankment height is less than 1.5m, very large post-construction settlements are anticipated. The calculated post construction settlements do not meet the performance requirements outlined in Section 3.2.5.2.6.3 of the RFQ:

- Maximum of 25mm of post construction settlement within 30m of the abutments
- Maximum of 50mm beyond 30m of the abutments
- The construction methods that include 3m of surcharge (Configurations A to B and C to D) result in post construction settlements (20 years) ranging from 280mm to 330mm within 220m of the SBL abutment and from 170mm to 280mm within 190m of the NBL abutment
- The construction methods that include the preloading of the foundation soils with sand fill up to the top of pavement elevation and subsequent replacement of the sand fill with EPS fill (Configurations D to E) result in post construction settlements (20 years) ranging from 140mm to 200mm within 220m of the SBL abutment and from 100mm to 170mm within 190m of the NBL abutment
- The construction method that includes construction of the embankments using EPS fill (Configuration E) without preloading, surcharging or sub-excavation of the foundation soils, result in post construction settlements (20 years) ranging from 70mm to 100mm within 220m of the SBL abutment and from 30mm to 120mm within 190m of the NBL abutment. Within 50m of the abutments the predicted post construction settlement is 85mm.
- The tables included in Appendix H show the variation of the parameter  $R'_s$  with depth.  $R'_s$  is equal to the over consolidation ratio minus one (OCR-1). The analysis of the  $R'_s$  indicates that the use of preloading and surcharge associated with the construction methods considered herein result in relatively low over consolidation of the silty clay within 220m to 190m of the SBL and NBL, respectively. This is due to the large depth of the silty clay deposit compared to the embankment width which results in relatively small increment of vertical stresses due to surcharging at depth.

## 10.5 Lateral Displacements at the Abutment Piles

For monitoring purposes and preliminary verification of the structural capacity of the abutment piles, the maximum long-term lateral deflection along the pile should be assumed to be equal to 20% (Ladd<sup>3</sup>) of the post construction settlement of the approach embankment at the centre of the silty clay layer, at approximate EL. 200. The lateral deflections can be assumed to decrease to zero above (EL. 215) and below (EL.180) the point of maximum deflection. For instance, if the embankment configuration E (EPS fill

<sup>3</sup> Ladd, C.C. (1991). A Stability Evaluation During Staged Construction, ASCE Journal of Geotechnical Engineering, Vol.17, No.4, 1991



without preloading and without sub-excavation) is adopted behind the abutments, the pile capacity should be verified for a maximum lateral displacement of 17mm (20% of 85mm) at EL200 and pinned at EL.215 and EL.180. If the native soils are subexcavated to 1.5m depth and replaced with EPS (Alternative E with subexcavation) the lateral deflection of the piles are expected to be negligible.

After selection of the pile types and if the anticipated lateral deflections above are critical for the structural performance of the piles we will be pleased to carry out a more detailed analysis of the soil-pile interaction.

## **10.6 Analysis of Design Alternatives**

### **10.6.1 General**

The sections above presented embankment design alternatives that include:

- Different materials utilized for the embankment construction (sand fill, rock fill and EPS)
- Preloading of the foundation soils including 3m surcharge above the top of pavement elevation (Configurations A and C) or construction of a sand fill embankment to the top of pavement (Configuration D) followed by removal of the sand fill and replacement with EPS fill (Configuration E). These methods aim at reducing post construction settlements by increasing the effective stresses in the foundation soils and over consolidating the silty clay.
- Construction of the embankment to its final configuration using EPS fills (Configuration E). This alternative aims at maintaining the effective stresses in the foundation soils low (possibly below the pre-consolidation pressures in the silty clay deposit) and to minimize disturbance of the silty clay, which is sensitive and structured

Due to the fact that the over-consolidation ratio values achieved in the silty clay with 3m surcharge and preloading with sand fill are relatively low, Configurations A and C are associated with large post-construction settlements. The use of more than 3m surcharge is not considered effective due to the large depth to the top of the silty clay layer.

Configuration E is effective in reducing post-construction settlements although the settlement values do not meet MTO's performance requirement. MTO's performance requirements can be met if Configuration E is implemented in conjunction with sub-excavation of the foundation soils to 1.5m depth within the footprint of the embankments and replacement with EPS.

In addition to the design alternatives described above, lengthening the bridge to reduce settlements due to secondary consolidation has been considered. This alternative would only be effective if the bridges were extended to approximate Station 10+080 in the Dill TWP. This would result in bridge lengths 260m to 285m longer than proposed in the current design, which is not considered economically feasible, as shown in Appendix J.

Lowering the vertical alignment of the highway in order to reduce the approach embankment height is beneficial from the geotechnical point of view. It is understood, however, that this alternative has been analysed and is considered not acceptable.

#### 10.6.2 Embankment Design Recommendations

The analysis presented herein shows that, from an economic point of view, it is not feasible to design the approach embankments for long term performance requirements outlined in MTO's RFQ. A compromise between cost and performance can be achieved by designing the north approach embankment as follows:

##### SBL Embankment

- From St.10+550 (North Abutment) to Station 10+590 (within 40m of the abutment)
  - Sub-excavate the native soils within the embankment footprint to 1.5m below original ground surface.
  - Install 0.3m thick drainage blankets within the footprint of the embankment. Drainage blankets are not required beyond the footprint of the embankment
  - Install wick drains in a triangular pattern spaced at 1.5m within and 5m beyond the footprint of the embankment to EL.175 (sandy silt layer underlying the compressible soils)
  - Construct the approach embankment in one stage to its final configuration using EPS fill to the underside of the pavement structure, protected with polyethylene sheeting and minimum of 1m of sand fill (Side slopes: 2H:1V)
  - Wait for dissipation of excess pore pressures in the foundation soils (4 months)
  - Install abutment piles and construct abutments
  - Finalize embankment construction
- From Station 10+590 to Station 10+010 – Dill TWP (40m to 220m north of the abutment)
  - Install wick drains in a triangular pattern spaced at 1.5m within and 5m beyond the footprint of the embankment to EL.175 to EL.200 (silt layer or bedrock underlying the compressible soils). Drainage blankets are not required in this area.
  - Construct the embankment in one stage using rock fill to 1m below the pavement structure (side slope: 1.25H:1V) and using sand fill (SSM: side slope 1.5H:1V) above the rock fill to 3m above the top of pavement. This zone will include a transition zone where the underside of the EPS fill will

be sloped upwards at 10H:1V (starting at Station 10+590) and will be underlain by rock fill. A 1m thick layer of Granular A should be installed at the interface between the EPS and underlying rock fill. The rock fill surface should be “chinked” to reduce loss of Granular A into the rock fill.

- Wait for dissipation of excess pore pressures in the foundation soils (4 months)
- Remove surcharge and finalize embankment construction
- From Station 10+010 to Station 10+070 Dill TWP (220m to 250m north of the abutment)
  - Construct embankment to its final configuration using rock fill (side slope: 1.25H:1V) and sand (side slope: 2H:1V) to the top of pavement. There is no need for preloading or surcharging in this area
  - Wait for dissipation of excess pore pressures in the foundation soils (4 months)
  - Remove surcharge and finalize embankment construction

#### NBL Embankment

- From St.10+580 (North Abutment) to Station 10+620 (within 40m of the abutment)
  - Sub-excavate the native soils within the embankment footprint to 1.5m below original ground surface.
  - Install 0.3m thick drainage blankets within the footprint of the embankment. Drainage blankets are not required beyond the footprint of the embankment
  - Install wick drains in a triangular pattern spaced at 1.5m within and 5m beyond the footprint of the embankment to EL.180 (sandy silt layer underlying the compressible soils)
  - Construct the embankment in one stage to its final configuration using EPS fill to the underside of the pavement structure, protected with polyethylene sheeting and minimum of 1m of sand fill (side slopes: 2H:1V)
  - Wait for dissipation of excess pore pressures in the foundation soils (4 months)
  - Install abutment piles and construct abutments
  - Finalize embankment construction

- From Station 10+620 to Station 10+010 – Dill TWP (40m to 190m north of the abutment)
  - Install wick drains in a triangular pattern spaced at 1.5m within the footprint of the embankment to EL.180 to EL.205 (silt layer or bedrock underlying the compressible soils). Drainage blankets are not required in this area.
  - Construct the embankment in one stage using rock fill (1.25H:1V) to 1m below the pavement structure and using sand fill (SSM: 1.5H:1V side slope) above the rock fill to 3m above the top of pavement. This zone will include a transition zone where the underside of the EPS fill will be sloped upwards at 10H:1V (starting at Station 10+590) and will be underlain by rock fill. A 1m thick layer of Granular A should be installed at the interface between the EPS and underlying rock fill. The rock fill surface should be “chinked” to reduce loss of Granular A into the rock fill.
  - Wait for dissipation of excess pore pressures in the foundation soils (4 months)
  - Remove surcharge and finalize embankment construction
- From Station 10+010 to Station 10+070 Dill TWP (190m to 250m north of the abutment)
  - Construct embankment to its final configuration using rock fill (1.25H:1V) and sand (2H:1V) to the top of pavement. There is no need for preloading or surcharging in this area
  - Wait for dissipation of excess pore pressures in the foundation soils (4 months)
  - Remove surcharge and finalize embankment construction

The approach embankment should be wide enough at the top to compensate for loss of width associated with large settlements.

Embankment construction should be in accordance with OPSS 206, as amended by Special Provision “Amendment to OPSS 206, December 1993”, dated November 2002.

Earth fill embankment slopes must be provided with erosion protection in accordance with OPSS 572.

The design proposed above was based on assumptions that should be confirmed with a geotechnical instrumentation and monitoring program implemented during construction of the embankments. The monitoring program is considered critical to reduce the risk of instability of the embankment during construction and to establish the appropriate time to remove the surcharge and start installation of the abutment piles.

Non-Standard Specifications (NSSP) for granular blanket and wick drains have been included in Appendix G. It should be noted that granular blankets are not required for drainage purposes if the embankment material is rock fill.

An analysis of design alternatives, including long-term performance and costs, is presented in Appendix J.

## 11 SEISMIC CONSIDERATIONS

### 11.1 Seismic Design Parameters

The site is treated as lying in Seismic Zone 1 and the following seismic parameters should be used for design (Table 3.1.7 of the CHBDC: Sudbury):

- Velocity Related Seismic Zone: 1
- Zonal Velocity Ratio: 0.05
- Acceleration Related Seismic Zone: 1
- Zonal Acceleration Ratio (PHA): 0.05g

The Soil Profile Type at this site is classified as Type I at the south abutments, where the bedrock is at surface or shallow and Type III elsewhere. According to Table 4.4.6.1 of the CHBDC these soil types are associated with Site Coefficient values (also referred to as ground motion amplification factor) of 1 and 1.5, respectively. Therefore a Peak Horizontal Ground Acceleration (PHA) at ground surface of 0.05 and 0.075g (where g is the gravity acceleration) should be used for design of the bridge.

### 11.2 Liquefaction Evaluation

A comprehensive review of the use of the piezocone test results and its application for this site is presented in Appendix E. In summary, although the data shows that the sand/silt deposit is compressible locally and can undergo significant strain softening in simple shear loading, the settlements and lateral displacements associated with a seismic event are anticipated to be negligible.

### 11.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause C4.6.4 of the CHBDC 2000 retaining structures should be designed using active ( $K_{AE}$ ) and passive earth pressure ( $K_{PE}$ ) coefficients that include earthquake loading. The following design parameters should be used to calculate  $K_{AE}$  and  $K_{PE}$  according to the CHBDC:

$\phi'$  = angle of internal friction of backfill

$\phi'$  = 35° for OPSS Granular A or Granular B Type II

- $\phi'$  = 32° for OPSS Granular B Type I
- $\phi'$  = 42° for Rock fill
- $k_h$  = horizontal acceleration coefficient
- $k_h$  = 0.5\*PHA for yielding structures (integral abutments)
- $k_h$  = 1.5\*PHA for non-yielding structures (rigid retaining walls and abutments founded on battered piles)

Recommended values are shown in Tables 10.8 and 10.9.

## 12 STATIC EARTH PRESSURES

Earth pressures acting on the structure for fully drained condition should be computed in accordance with the CHBDC, which are generally given by the expression:

$$P_h = K*(\gamma h + q)$$

Where:

$P_h$  = horizontal pressure on the wall at depth "h"(kPa)

$K$  = earth pressure coefficient (see below)

$\gamma$  = unit weight of retained soil (typically 21 kN/m<sup>3</sup>)

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added to the lateral earth pressure above. The magnitude should be 12kPa at the top of fill and decreasing to 0kPa at a depth of 2.0 m for Granular B Type I or 1.7 m for Granular A or Granular B Type II.

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 10.10 below.

**Table 10.10 – Static Earth Pressure Coefficients**

Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		Rock Fill $\phi = 42^\circ, \gamma = 19 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.47*	0.2	.30*
At rest (Restrained Wall)	0.43	-	0.47	-	.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	5.0	-

(\*) For wing walls

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass. However, the use of Granular “B” Type I may be restricted if the approach embankment consist of rock fill.

The factors in the table above are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the Canadian Highway Bridge Design Code.

### 13 CONSTRUCTION CONCERNS AND CONCLUDING REMARKS

Potential construction concerns include, but are not necessarily limited to:

- Wick drain obstructions in the upper 12m to 15m of the foundation soils and installation carried out late in the fall and without enough fill cover for protection against freezing
- Stability of the temporary forward slope of SBL north approach embankment (unless EPS fill is used for the construction of the embankment). Staged construction or a stabilizing berm is required.

- The temporary access roads for the construction of the pier foundations located in the swamp have potential for instability and may require the use of geogrid and other stabilization measures. The temporary access road embankments should be designed by an experienced engineer.
- The presence of a deep layer of peat may pose difficulties for the installation of some of the pier foundations. Sheet pile enclosure and dewatering may be required for construction of the foundation pile caps.
- In light of the thick deposit of compressible soils, the north abutment pile foundations will be subjected to large downdrag loads unless Alternative E with sub-excavation of the native soils to 1.5m depth is implemented within 40m of the abutments. Appropriate measures such as bitumen coating of the piles or installing them inside a steel casing may be required to mitigate the downdrag loads.
- Piers 2 and 3 will be construction relatively close to CN Rail's track. The construction activities during the pile installation including pile-driving, may cause settlement of the track. CN rail should be notified of the potential problem and should be ready to resurface the track on a regular basis during construction. A monitoring program should be designed for CN Rails embankments for track settlement control during and shortly after the end of construction.
- Presence of unfavourable discontinuities in the bedrock at the SBL and NBL south abutments may require stabilization measures if spread footings are used.
- The results of the approach embankment monitoring program will control the construction schedule. Although not anticipated, there is a risk that the pore pressure dissipation in the foundation soils will be slower than anticipated. If this situation occurs, depending on the embankment configuration selected for design behind the north abutments, the removal of surcharge and installation of piles may be delayed, which may impact the overall construction schedule. A detailed and regular analysis of the monitoring program during construction is considered critical to:
  - Reduce potential of an embankment failure, particularly at the SBL forward slope
  - Reduce the risk of a premature removal of the surcharge
  - Reduce the risk of installing the abutment piles too early

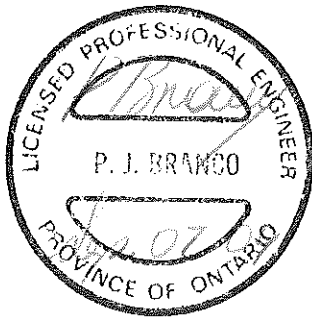
During construction, the Contract Administrator should employ experienced high complexity geotechnical staff to implement the geotechnical monitoring program and to observe construction activities related to foundation construction.

The design recommendations provided in this report were based on a comprehensive geotechnical investigation and well known modelling tools. Notwithstanding this fact, it should be recognized that the conditions at the proposed bridge site and north approach embankments are extremely complex from the geotechnical viewpoint. The presence of very thick compressible clayey deposits, which are locally very sensitive and varved, adds significant uncertainty to the prediction of the performance of the structures and embankment proposed in this project. As a result, the



settlement values predicted in this report, in particular the post construction settlements, should be interpreted simply as an index that should be used for the selection of an embankment configuration and construction method that would reduce the potential for the occurrence of large post construction settlements. In addition, it is considered critical that the construction contract includes clauses that allow for a flexible construction schedule to allow for delays associated with dissipation of excess pore pressures in the foundation soils lower than anticipated.

Engineering analysis and report prepared by:



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Project Engineer, Principal



Report Reviewed by:  
P. K. Chatterji, Ph.D., P.Eng.  
Review Principal

## OEDOMETER TEST RESULTS

Borehole	BH-14N	BH-17S	BH-15N	BH-18S
Sample	ST#1	ST#1	ST#2	ST#2
Depth (m)	20.1	24.15	33.8	38.4
Elevation (m)	204.2	199.95	191.8	186.0
Soil Type	CL-CI	CL-CI	CH	CI-CH
% Clay	40%	45%	77%	65%
W.C. (%)	32.3%	34.1%	50.4%	45.2%
Unit Weight (g) (kN/m <sup>3</sup> )	18.91	18.62	16.98	17.36
Specific Gravity (G)	2.78	2.76	2.8	2.78
Initial Void Ratio	0.908	0.949	1.432	1.281
In situ effective vertical stress	210	240	322	354
Preconsolidation Pressure (p')	210	260	322	400
Compression Index (Cc)	0.23	0.32	0.63	0.66
Recompression Index (Cr)	0.02	0.04	0.08	0.06

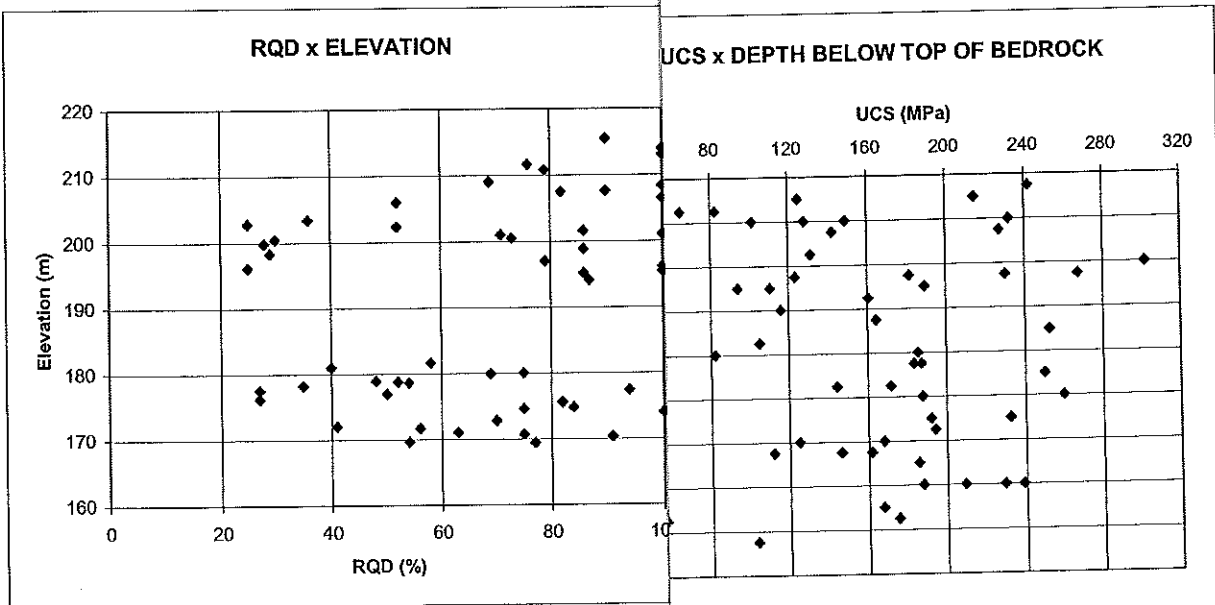
TRIAXIAL TEST RESULTS

Borehole	BH-13N	BH-16S
Sample	ST#2	ST#2
Depth (m)	24.075	36.275
Elevation (m)	199.925	187.125
Soil Type	CH	CH
% Clay	83%	70%
W.C. (%)	46.3%	48.8%
Unit Weight (g) (kN/m3)	17.45	17.6
In situ effective vertical stress	240	340
Pore Pressure Coefficient (Af)		
at Consolidation Pressure =		
50kPa	0.06	-
100kPa	0.03	0.41
150kPa	0.51	-
180kPa	-	0.4
250kPa	-	0.54
Cohesion (c') (kPa)	13	0
Friction Angle (φ') (°)	31	31

Triaxial Results

TABLE 5.2

	BH	Ground Surface EL. (m)	Top of Bedrock Elevation	Run #1			Run #6				
				Elevation (m)	Depth below T.O.B/R (m)	RQD (%)	UCS MPa	Elevation (m)	Depth below T.O.B/R (m)	RQD (%)	UCS MPa
SBL Bridge	BH-6S	219.0	216.0	215.5	0.5	90	-	-	-	-	-
	BH-10S	219.2	212.6	212.5	0.1	-	-	-	-	-	-
	BH-11S	219.2	201.4	201	0.4	100	56.3	-	-	-	-
	BH-12S	219.3	203.5	203.4	0.1	36	161.72	200.4	3.1	30	175.28
	BH-13S	219.1	181.3	181	0.3	4	-	-	-	-	-
	BH-14S	219.0	182.4	181.6	0.8	5	-	-	-	-	-
	BH-15S	223.4	171.9	171.7	0.2	5	-	-	-	-	-
NBL Bridge	BH-16S	223.4	172.4	-	-	-	-	-	-	-	-
	BH-6N	219.0	208.4	207.5	0.9	8	168	-	-	-	-
	BH-7N	219.2	209.3	209	0.3	6	-	-	-	-	-
	BH-8N	219.2	196.5	-	-	-	-	-	-	-	-
	BH-9N	219.2	198.0	197	1.0	7	-	-	-	-	-
	BH-10N	220.3	179.1	178.2	0.9	3	-	-	-	-	-
	BH-11N	221.0	179.0	178.6	0.4	5	-	-	-	-	-
	BH-12N	224.2	176.2	176	0.2	-	-	-	-	-	-
	BH-13N	224.0	177.9	177.4	0.5	9	-	-	-	-	-



Notes: RQD : Rock Quality Designation  
UCS: Uniaxial Compression Strength

TABLE 5.4

### PIEZOMETRIC HEAD OBSERVATIONS

Test Hole	Date	Tip Elevation (m)	Ground Surface Elevation (m)	Piezometric Head Elevation (m)	Depth Below G.Surface (m)	Remarks
BH-11S	1-Jun-04	201.4	219.2	219.2	0.0	Swamp
BH-13S	1-Jun-04	181.3	219.1	220.2	-1.1	Above GS
BH-15S	1-Jun-04	172.5	223.4	220.3	3.1	
BH-6N	8-Jun-04	207.1	219	219.0	0.0	Swamp
BH-8N	8-Jun-04	194.9	219.2	219.2	0.0	Swamp
BH-10N	8-Jun-04	179.0	220.3	218.2	2.1	
BH-12N	8-Jun-04	174.5	224.2	220.4	3.8	
BH612-8M	16-Apr-03	209.0	225.6	218.1	7.5	Peto's
	2-Jun-04			218.7	6.9	
BH612-20S	11-Apr-03	207.0	223.7	217.6	6.1	Peto's
	2-Jun-04			218.3	5.4	
	11-Apr-03	213.0	223.7	217.2	6.5	Peto's
	2-Jun-04			221.5	2.2	
BH612-14N	29-Apr-03	208.0	222.3	216.4	6.0	Peto's
	2-Jun-04			217.7	4.6	
	29-Apr-03	215.5	222.3	216.8	5.5	Peto's
	2-Jun-04			218.1	4.3	

**TABLE 7.3 - COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT**

Foundation Element	Driven Piles	Spread Footing	Caissons
SBL and NBL South Abutments	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>• High capacity for piles seating on bedrock</li> <li>• Compatible with Integral Abutment design</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>• Short piles have to be socketed in bedrock for required lateral flexibility</li> <li>• Probably most costly and spread footings</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>• High values of geotechnical resistance are available on bedrock</li> <li>• Costs lower than driven piles</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>• Precludes the use of integral abutment</li> <li>• May require benching or the bedrock surface</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>• High bearing capacity on bedrock</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>• Precludes use of Integral Abutments</li> <li>• Not practical due to shallow bedrock</li> </ul>
Pier 1S	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>• High capacity for piles seating on bedrock</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>• Depending on the foundation elevation and lateral resistance requirements, piles may have to be socketed into bedrock</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>• High values of geotechnical resistance are available on bedrock</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>• Relatively deep and sloping bedrock</li> <li>• Difficult to maintain excavation dry during construction of footing</li> <li>• High groundwater table: combination of watertight shoring and dewatering will be required</li> <li>• Costs probably higher than driven piles</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>• High bearing capacity on bedrock</li> <li>• High groundwater table: dewatering will be required</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>• Installation through cohesionless soils and high groundwater table: difficult installation and difficult quality control.</li> <li>• Higher cost than driven piles</li> </ul>
Piers 2S, 3S, 1N, 2N, 3N and SBL and NBL North Abutments	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>• High capacity for piles seating on bedrock</li> <li>• Relatively straightforward installation</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>• Long term settlements may require special provisions at the North Abutment piles to reduce downdrag forces</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>• Lower cost than piled foundation</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>• Not feasible due to large immediate and long term settlements</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>• High bearing capacity on bedrock</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>• Installation through cohesionless soils and high groundwater table: difficult installation and difficult quality control.</li> <li>• Precludes use of Integral Abutments</li> <li>• Higher cost than driven piles</li> </ul>

HIGHWAY 69 - FOUR LANING - CNR CROSSING NORTH OF ESTAIRE  
SOIL PROPERTIES FOR STABILITY AND SETTLEMENT ANALYSIS - SOUTHBOUND LANE BRIDGE

Location	Soil Layer	Depth Interval		Unit Weight (kN/m3)	Undrained Shear Strength		Drained Shear Strength		Shear Strength at Pile/Soil Interface (Beta)	Poisson's Ratio	Young's Modulus (MPa)	Compression Ratio		Over Consolidation Ratio (OCR)	Coeff. Of Consolidation (m2/y)				Secondary Compression Ratio $C_{\alpha}/(1+e_0)$	
		From (m)	To (m)		Cohesion (kPa)	Friction Angle (deg)	Cohesion (kPa)	Friction Angle (deg)				$C_c/(1+e_0)$	$C_r/(1+e_0)$		Cv		Ch			
North Abutment  10+550 to 10+600	Rock Fill	top of fill	0.5	19	---	---	0	42	0.3	0.30	150	---	---	---	---	---	---	---	---	
	Sand & Silt (NP)	0.5	5	19	---	---	0	32	0.3	0.35	30	---	---	---	---	---	---	---	---	
	Clayey Silt	5	8	19	60	---	0	30	0.25	0.40	18	0.13	0.013	2	100	75	200	150	0.0026	
	Sand & Silt (NP)	8	13	19	---	---	0	32	0.3	0.35	30	---	---	---	---	---	---	---	---	
	Clay (CL)	13	25	18.5	45	---	0	28	0.25	0.45	13.5	0.22	0.022	1.0 to 1.1	40	30	80	60	0.0044	
	Clay (CH)	25	40	17.5	90	---	0	26	0.3	0.45	27	0.25	0.025	1.0 to 1.2	25	20	25	20	0.005	
	Clayey Silt (CL-ML)	40	47	19	80	---	0	30	0.3	0.40	24	0.18	0.018	1.0 to 1.1	100	75	200	150	0.0036	
	Silt & Sand (NP)	47	51	21	---	---	0	32	0.3	0.35	30	---	---	---	---	---	---	---	---	
Bedrock	51	>51	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	
North Approach-1  10+600 to 10+675	Rock Fill	top of fill	0.5	19	---	---	0	42	0.3	0.30	150	---	---	---	---	---	---	---	---	---
	Sand & Silt (NP)	0.5	8	19	---	---	0	32	0.3	0.35	30	---	---	---	---	---	---	---	---	---
	Clayey Silt	8	12	19	60	---	0	30	0.25	0.40	18	0.13	0.013	1.5	100	75	200	150	0.0026	
	Sand & Silt (NP)	12	14	19	---	---	0	30	0.3	0.35	30	---	---	---	---	---	---	---	---	---
	Clay (CL)	14	24	18.5	45	---	0	28	0.25	0.45	13.5	0.22	0.022	1.0 to 1.1	40	30	80	60	0.0044	
	Clay (CH)	24	41.5	17.5	90	---	0	26	0.3	0.45	27	0.25	0.025	1.0 to 1.2	25	20	25	20	0.005	
	Silty Clay (CL-ML)	41.5	45.5	19	80	---	0	30	0.3	0.40	24	0.18	0.018	1.0 to 1.1	100	75	200	150	0.0036	
	Silt & Sand (NP)	45.5	51	21	---	---	0	32	0.3	0.35	30	---	---	---	---	---	---	---	---	---
Bedrock	51	>51	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	
North Approach-2 10+675 to Dill TWP 10+010	Rock Fill	top of fill	0.5	19	---	---	0	42	0.3	0.30	150	---	---	---	---	---	---	---	---	---
	Sand	0.5	4	19	---	---	0	30	0.3	0.35	30	---	---	---	---	---	---	---	---	---
	Silty Clay (CL-ML)	4	7	19	50	---	0	29	0.25	0.45	15	0.18	0.018	1.0 to 1.5	50	40	100	75	0.0036	
	Sand	7	10.5	19	---	---	0	32	0.3	0.35	30	---	---	---	---	---	---	---	---	---
	Clay (CL)	10.5	22	18.5	45	---	0	28	0.25	0.45	13.5	0.22	0.022	1.0 to 1.1	40	30	80	60	0.0044	
	Clay CH	22	38	17.5	70	---	0	26	0.3	0.45	21	0.25	0.025	1.0 to 1.2	25	20	25	20	0.005	
	Silty Clay (CL-ML)	38	43	19	80	---	0	30	0.3	0.40	24	0.20	0.020	1.0 to 1.1	100	75	200	150	0.004	
	Bedrock	43	>43	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
North Approach-3 Dill TWP 10+010 to 10+040	Rock Fill	top of fill	0.5	19	---	---	0	42	0.3	0.30	150	---	---	---	---	---	---	---	---	---
	Sand & Silt (NP)	0.5	3.5	19	---	---	0	30	0.3	0.35	30	---	---	---	---	---	---	---	---	---
	Clay	3.5	6	19	50	---	0	29	0.25	0.45	15	0.18	0.018	1.0 to 1.5	50	40	100	75	0.0036	
	Sand	6	7.5	19	---	---	0	32	0.3	0.35	30	---	---	---	---	---	---	---	---	---
	Clay (CL)	7.5	8.5	18.5	45	---	0	28	0.25	0.45	13.5	0.22	0.022	1.0 to 1.1	40	30	80	60	0.0044	
	Bedrock	8.5	>8.5	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---

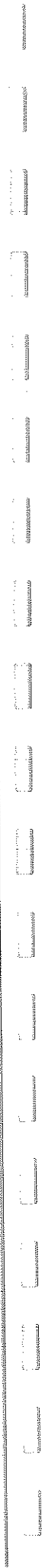
Notes: O.C.: Over Consolidated Soil  
N.C.: Normally Consolidated Soil

HIGHWAY 69 - FOUR LANING - CNR CROSSING NORTH OF ESTAIRE  
SOIL PROPERTIES FOR STABILITY AND SETTLEMENT ANALYSIS - NORTHBOUND LANE BRIDGE

Location	Soil Layer	Depth Interval		Unit Weight (kN/m3)	Undrained Shear Strength		Drained Shear Strength		Shear Strength at Pile/Soil Interface (Beta)	Poisson's Ratio	Young's Modulus (MPa)	Compression Ratio		Over Consolidation Ratio (OCR)	Coeff. Of Consolidation (m2/y)				Secondary Compression Ratio $C_{\alpha}/(1+e_0)$
		From (m)	To (m)		Cohesion (kPa)	Friction Angle (deg)	Cohesion (kPa)	Friction Angle (deg)				$C_c/(1+e_0)$	$C_r/(1+e_0)$		Cv		Ch		
North Abutment 10+580 to 10+620	Rock Fill	top of fill	0.5	19	---	---	0	42	0.3	0.30	150	---	---	---	---	---	---	---	---
	Sand & Silt (NP)	0.5	13	19	---	---	0	32	0.3	0.35	30	---	---	---	---	---	---	---	---
	Clay (CL)	13	25	18.5	45	---	0	28	0.25	0.45	13.5	0.22	0.022	1.0 to 1.1	40	30	80	60	0.0044
	Clay (CH)	25	40	17.5	90	---	0	26	0.3	0.45	27	0.25	0.025	1.0 to 1.2	25	20	25	20	0.005
	Clayey Silt (CL-ML)	40	42	19	80	---	0	30	0.3	0.40	24	0.18	0.018	1.0 to 1.1	100	75	200	150	0.0036
	Silt & Sand (NP)	42	48	21	---	---	0	32	0.3	0.35	30	---	---	---	---	---	---	---	---
North Approach-1 10+620 to 10+675	Bedrock	48	>51	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
	Rock Fill	top of fill	0.5	19	---	---	0	42	0.3	0.30	150	---	---	---	---	---	---	---	---
	Sand & Silt (NP)	0.5	14	19	---	---	0	32	0.3	0.35	30	---	---	---	---	---	---	---	---
	Clay (CL)	14	25	18.5	45	---	0	28	0.25	0.45	13.5	0.22	0.022	1.0 to 1.1	40	30	80	60	0.0044
	Clay (CH)	25	42	17.5	90	---	0	26	0.3	0.45	27	0.25	0.025	1.0 to 1.2	25	20	25	20	0.005
	Silt (ML-NP)	42	45.5	19	---	---	0	32	0.3	0.40	30	---	---	---	---	---	---	---	---
North Approach-2 10+675 to Dill TWP 10+010	Silt & Sand (NP)	45.5	49	21	---	---	0	32	0.3	0.35	30	---	---	---	---	---	---	---	---
	Bedrock	49	>51	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
	Rock Fill	top of fill	0.5	19	---	---	0	42	0.3	0.30	150	---	---	---	---	---	---	---	---
	Sand	0.5	4	19	---	---	0	30	0.3	0.35	30	---	---	---	---	---	---	---	---
	Silty Clay (CL-ML)	4	7	19	50	---	0	29	0.25	0.45	15	0.18	0.018	1.0 to 1.5	50	40	100	75	0.0036
	Sand	7	10	19	---	---	0	32	0.3	0.35	30	---	---	---	---	---	---	---	---
North Approach-3 Dill TWP 10+010 to 10+040	Clay (CL)	10	20	18.5	45	---	0	28	0.25	0.45	13.5	0.22	0.022	1.0 to 1.1	40	30	80	60	0.0044
	Clay CH	20	29	17.5	70	---	0	26	0.3	0.45	21	0.25	0.025	1.0 to 1.2	25	20	25	20	0.005
	Silty Clay (CL-ML)	29	34	19	80	---	0	30	0.3	0.40	24	0.20	0.020	1.0 to 1.1	100	75	200	150	0.004
	Bedrock	34	>34	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
	Rock Fill	top of fill	0.5	19	---	---	0	42	0.3	0.30	150	---	---	---	---	---	---	---	---
	Sand & Silt (NP)	0.5	5.5	19	---	---	0	30	0.3	0.35	30	---	---	---	---	---	---	---	---
	Clay	5.5	7.5	19	50	---	0	29	0.25	0.45	15	0.18	0.018	1.0 to 1.5	50	40	100	75	0.0036
	Clay (CL)	7.5	14.4	18.5	45	---	0	28	0.25	0.45	13.5	0.22	0.022	1.0 to 1.1	40	30	80	60	0.0044
	Bedrock	14.4	>14.4	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---

Notes: O.C.: Over Consolidated Soil  
N.C.: Normally Consolidated Soil





64-15-Embankment Configurations.XLS

EMBANKMENT CONFIGURATIONS AND MATERIALS

Initial Configuration (Prior to Removal of Surcharge)		Final Configuration (After Removal of Surcharge)	
A	Rock fill embankment and 3m surcharge (sand)	B	Rock fill embankment
C	SSM embankment with 3m surcharge (sand)	D	SSM embankment
D	SSM embankment (no surcharge)	E	EPS fill with 1m sand envelope
E	EPS fill with 1m sand envelope (no surcharge)	E	EPS fill with 1m sand envelope

Settlements due to Primary Consolidation  
SBL North Approach Embankment

Notes: (\*) : Due to primary consolidation only  
(\*\*) : Time after the end of construction to the top of the embankment for Intermediate Configuration

Embankment Configurations		No Wick Drains			With Wick Drains			
		Ultimate Settlement Due to Primary Consolidation (m)	Time (**) required for 98% Consolidation for Intermediate Configuration	Time (**) required for removal of surcharge and stabilization of settlements due to primary consolidation	Time Required for 98% Consolidation For Intermediate Configuration			
					Wick drain spacing (triangular pattern)			
					s = 1.5m	s = 1.75m	s = 2m	
North Abutment 10+550 to 10+600	Normally Consolidated Silty Clay					4 months	5 months	6 months
	Intermediate	A	1.05	> 4years	2 to 3 years			
	Final	B	0.83	-	-			
	Intermediate	C	1.24	> 4years	3 to 4 years			
	Final	D	1.03	-	-			
	Intermediate	D	1.03	> 4years	< 1 year			
	Final	E	0.34	-	-			
	Intermediate	E	0.34	> 4years	-			
	Final	E	0.34	-	-			
	Over Consolidated Silty Clay							
	Intermediate	A	0.54	> 4years	1 to 2 years			
	Final	B	0.34	-	-			
	Intermediate	C	0.71	> 4years	3 to 4 years			
	Final	D	0.52	-	-			
North Approach-1 10+600 to 10+675	Normally Consolidated Silty Clay					4 months	5 months	6 months
	Intermediate	A	0.82	> 4years	1 to 2 years			
	Final	B	0.6	-	-			
	Intermediate	C	0.96	> 4years	2 to 3 years			
	Final	D	0.74	-	-			
	Intermediate	D	0.74	> 4years	< 1 year			
	Final	E	0.28	-	-			
	Intermediate	E	0.28	> 4years	-			
	Final	E	0.28	-	-			
	Over Consolidated Silty Clay							
	Intermediate	A	0.36	> 4years	< 1 year			
	Final	B	0.2	-	-			
	Intermediate	C	0.47	> 4years	1 to 2 years			
	Final	D	0.29	-	-			
North Approach-2 10+675 to Dill TWP 10+010	Normally Consolidated Silty Clay					4 months	5 months	6 months
	Intermediate	A	1.12	> 4years	1 to 2 years			
	Final	B	0.84	-	-			
	Intermediate	C	1.3	> 4years	2 to 3 years			
	Final	D	1.04	-	-			
	Intermediate	D	1.04	> 4years	< 1 year			
	Final	E	0.38	-	-			
	Intermediate	E	0.38	> 4years	-			
	Final	E	0.38	-	-			
	Over Consolidated Silty Clay							
	Intermediate	A	0.63	> 4years	< 1 year			
	Final	B	0.39	-	-			
	Intermediate	C	0.79	> 4years	1 to 2 years			
	Final	D	0.54	-	-			
North Approach-3 Dill TWP 10+010 to Dill TWP 10+040	Normally Consolidated Silty Clay					1 month	1 month	1 month
	Intermediate	A	0.26	< 1 year	< 1 year			
	Final	B	0.18	-	-			
	Intermediate	C	0.28	< 1 year	< 1 year			
	Final	D	0.21	-	-			
	Intermediate	D	0.21	< 1 year	< 1 year			
	Final	E	0.09	-	-			
	Intermediate	E	0.09	< 1 year	-			
	Final	E	0.09	-	-			
	Over Consolidated Silty Clay							
	Intermediate	A	0.18	< 1 year	< 1 year			
	Final	B	0.1	-	-			
	Intermediate	C	0.2	< 1 year	< 1 year			
	Final	D	0.13	-	-			

Settlements due to Primary Consolidation  
NBL North Approach Embankment

Notes: (\*) : Due to primary consolidation only  
(\*\*) : Time after the end of construction to the top of the embankment for Intermediate Configuration

No wicks					With Wick Drains			
Embankment Configurations			Ultimate Settlement Due to Primary Consolidation (m)	Time (**) required for 98% Consolidation for Intermediate Configuration	Time (**) required for removal of surcharge and stabilization of settlements due to primary consolidation	Time Required for 98% Consolidation For Intermediate Configuration		
						Wick drain spacing (triangular pattern)		
						s = 1.5m	s = 1.75m	s = 2m
North Abutment 10+580 to 10+620	Normally Consolidated Silty Clay					4 months	5 months	6 months
	Intermediate	A	0.88	> 4years	2 to 3 years			
	Final	B	0.7	-				
	Intermediate	C	1.05	> 4years	3 to 4 years			
	Final	D	0.88	-				
	Intermediate	D	0.88	> 4years	< 1 year			
	Final	E	0.27	-				
	Intermediate	E	0.27	> 4years	-			
	Final	E	0.27	-				
	Over Consolidated Silty Clay							
	Intermediate	A	0.5	> 4years	2 to 3 years			
	Final	B	0.35	-				
	Intermediate	C	0.66	> 4years	3 to 4 years			
	Final	D	0.5	-				
	Intermediate	D	0.5	> 4years	< 1 year			
	Final	E	0.07	-				
Intermediate	E	0.07	> 4years	-				
Final	E	0.07	-					
North Approach-1 10+620 to 10+675	Normally Consolidated Silty Clay					4 months	5 months	6 months
	Intermediate	A	0.66	> 4years	1 to 2 years			
	Final	B	0.47	-				
	Intermediate	C	0.77	> 4years	2 to 3 years			
	Final	D	0.59	-				
	Intermediate	D	0.59	> 4years	< 1 year			
	Final	E	0.21	-				
	Intermediate	E	0.21	> 4years	-			
	Final	E	0.21	-				
	Over Consolidated Silty Clay							
	Intermediate	A	0.28	> 4years	1 to 2 years			
	Final	B	0.18	-				
	Intermediate	C	0.36	> 4years	< 1 year			
	Final	D	0.25	-				
	Intermediate	D	0.25	> 4years	< 1 year			
	Final	E	0.04	-				
Intermediate	E	0.04	> 4years	-				
Final	E	0.04	-					
North Approach-2 10+675 to Dill TWP 10+010	Normally Consolidated Silty Clay					4 months	5 months	6 months
	Intermediate	A	0.82	> 4years	< 1 year			
	Final	B	0.55	-				
	Intermediate	C	1.01	> 4years	1 to 2 years			
	Final	D	0.67	-				
	Intermediate	D	0.67	> 4years	< 1 year			
	Final	E	0.28	-				
	Intermediate	E	0.28	> 4years	-			
	Final	E	0.28	-				
	Over Consolidated Silty Clay							
	Intermediate	A	0.47	> 4years	< 1 year			
	Final	B	0.26	-				
	Intermediate	C	0.64	> 4years	< 1 year			
	Final	D	0.36	-				
	Intermediate	D	0.36	> 4 years	< 1 year			
	Final	E	0.07	-				
Intermediate	E	0.07	> 4years	-				
Final	E	0.07	-					
North Approach-3 Dill TWP 10+010 to Dill TWP 10+040	Normally Consolidated Silty Clay					3 months	4 months	5 months
	Intermediate	A	0.49	< 2 years	< 1 year			
	Final	B	0.34	-				
	Intermediate	C	0.55	< 2 years	<1 year			
	Final	D	0.41	-				
	Intermediate	D	0.41	< 2 years	<1 year			
	Final	E	0.17	-				
	Intermediate	E	0.17	< 2 years	-			
	Final	E	0.17	-				
	Over Consolidated Silty Clay							
	Intermediate	A	0.48	< 2 years	< 1 year			
	Final	B	0.23	-				
	Intermediate	C	0.48	< 2 years	<1 year			
	Final	D	0.29	-				
	Intermediate	D	0.29	< 2 years	<1 year			
	Final	E	0.08	-				
Intermediate	E	0.08	< 2 years	-				
Final	E	0.08	-					

Settlements due to Secondary Consolidation  
SBL North Approach Embankment

Note: (\*) OCR in the Silty Clay deposit for the Final Configuration, after the removal of surcharge, if relevant

Embankment Configurations		Over Consolidation in the Silty Clay (*)		Post-Construction Settlements (mm)					
				Time after completion of Primary Consolidation					
Intermediate	Final	From	to	1 year	3 years	6 years	10 years	20 years	
North Abutment 10+550 to 10+600	Normally Consolidated Silty Clay								
	A	B	1.02	1.23	130	195	239	271	319
	C	D	1.03	1.21	137	203	247	281	328
	D	E	1.09	1.73	27	59	86	108	142
	E	E	1.00	1.00	77	136	179	213	259
	Over Consolidated Silty Clay								
	A	B	1.03	1.23	129	194	238	271	318
	C	D	1.03	1.21	136	202	246	280	327
	D	E	1.09	1.72	27	59	86	108	143
	E	E	1.00	1.20	25	45	59	70	85
North Approach 1 10+600 to 10+675	Normally Consolidated Silty Clay								
	A	B	1.03	1.22	116	178	219	251	296
	C	D	1.03	1.20	121	183	225	258	303
	D	E	1.06	1.41	53	96	129	155	194
	E	E	1.00	1.00	75	132	174	207	252
	Over Consolidated Silty Clay								
	A	B	1.03	1.21	116	177	219	251	296
	C	D	1.03	1.20	120	183	225	257	303
	D	E	1.06	1.40	53	96	129	155	195
	E	E	1.02	1.23	21	37	49	58	71
North Approach 2 10+675 to Dill TWP 10+010	Normally Consolidated Silty Clay								
	A	B	1.03	1.30	108	168	209	241	287
	C	D	1.03	1.27	114	175	217	250	296
	D	E	1.08	1.64	34	69	97	121	156
	E	E	1.00	1.00	77	136	180	213	260
	Over Consolidated Silty Clay								
	A	B	1.03	1.30	108	168	209	241	287
	C	D	1.03	1.27	114	175	217	250	296
	D	E	1.08	1.63	34	69	97	121	157
	E	E	1.00	1.20	29	52	68	81	99
North Approach 3 Dill TWP 10+010 to Dill TWP 10+040	Normally Consolidated Silty Clay								
	A	B	1.24	1.40	0	2	3	4	7
	C	D	1.22	1.36	0	2	3	5	7
	D	E	1.40	1.60	0	1	2	2	4
	E	E	1.00	1.00	6	11	15	18	22
	Over Consolidated Silty Clay								
	A	B	1.24	1.40	0	2	3	4	7
	C	D	1.22	1.36	0	2	3	5	7
	D	E	1.40	1.60	0	1	2	2	4
	E	E	1.00	1.20	2	4	5	6	7

Settlements due to Secondary Consolidation  
NBL North Approach Embankment

Note: (\*) OCR in the Silty Clay deposit for the Final Configuration, after the removal of surcharge, if relevant

Embankment Configurations		Over Consolidation in the Silty Clay (*)		Post-Construction Settlements (mm)					
				Time after completion of Primary Consolidation					
Intermediate	Final	From	to	1 year	3 years	6 years	10 years	20 years	
North Abutment 10+580 to 10+620	Normally Consolidated Silty Clay								
	A	B	1.03	1.23	109	164	201	230	270
	C	D	1.03	1.11	114	171	208	237	277
	D	E	1.12	1.45	16	39	60	78	105
	E	E	1.00	1.00	64	114	150	179	218
	Over Consolidated Silty Clay								
	A	B	1.03	1.12	109	164	202	230	270
	C	D	1.03	1.11	114	171	209	237	278
	D	E	1.12	1.45	16	39	60	78	106
	E	E	1.02	1.20	25	45	59	70	85
North Approach 1 10+620 to 10+675	Normally Consolidated Silty Clay								
	A	B	1.04	1.13	100	154	190	217	256
	C	D	1.03	1.13	104	158	194	223	262
	D	E	1.07	1.28	44	82	110	133	168
	E	E	1.00	1.00	64	113	149	176	215
	Over Consolidated Silty Clay								
	A	B	1.04	1.14	100	154	190	217	256
	C	D	1.03	1.13	104	158	195	223	262
	D	E	1.07	1.27	44	82	110	133	168
	E	E	1.06	1.20	5	13	21	27	38
North Approach 2 10+675 to Dill TWP 10+010	Normally Consolidated Silty Clay								
	A	B	1.05	1.36	56	94	122	144	176
	C	D	1.05	1.33	59	99	127	149	182
	D	E	1.08	1.46	30	59	82	100	129
	E	E	1.00	1.00	57	101	134	158	193
	Over Consolidated Silty Clay								
	A	B	1.05	1.36	56	94	122	144	176
	C	D	1.05	1.33	59	99	127	149	181
	D	E	1.08	1.45	30	60	82	101	129
	E	E	1.00	1.20	34	60	79	94	114
North Approach 3 Dill TWP 10+010 to Dill TWP 10+040	Normally Consolidated Silty Clay								
	A	B	1.15	1.31	4	10	16	21	30
	C	D	1.14	1.28	5	12	18	24	32
	D	E	1.26	1.48	1	4	7	11	16
	E	E	1.00	1.00	18	32	42	50	61
	Over Consolidated Silty Clay								
	A	B	1.15	1.31	4	10	16	21	30
	C	D	1.14	1.28	5	12	18	24	32
	D	E	1.26	1.48	1	4	7	11	16
	E	E	1.00	1.50	15	26	34	40	49

64-15-Seismic Earth Pressure Coefficients.xls

Standard Table for MTO's Reports:

Earth Pressure Coefficients for Earthquake Loading (CHBDC 2000)  
PHA = .05g

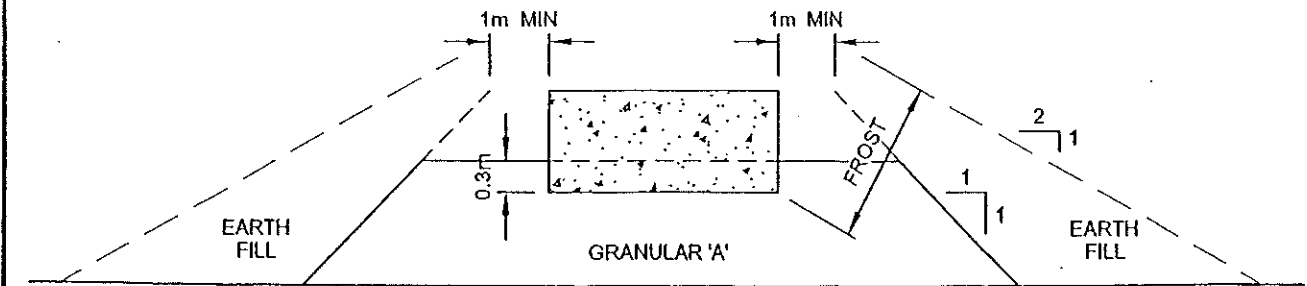
Condition	Earth Pressure Coefficient (K) for Earthquake Loading					
	Granular A or Granular B-II		Granular B - I		Rock Fill	
	$\phi=35^\circ$	$\delta=17^\circ$	$\phi=32^\circ$	$\delta=16^\circ$	$\phi=42^\circ$	$\delta=21^\circ$
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
	Yielding Wall					
	Active (KAE)	0.26	0.40	0.29	0.49	0.19
Passive (KPE)	7.05	63.53	5.66	34.75	14.40	-
	Non-Yielding Wall					
Active (KAE)	0.29	0.49	0.32	0.62	0.22	0.32
Passive (KPE)	6.77	62.17	5.43	33.97	13.88	-

64-15-Seismic Earth Pressure Coefficients-B.xls

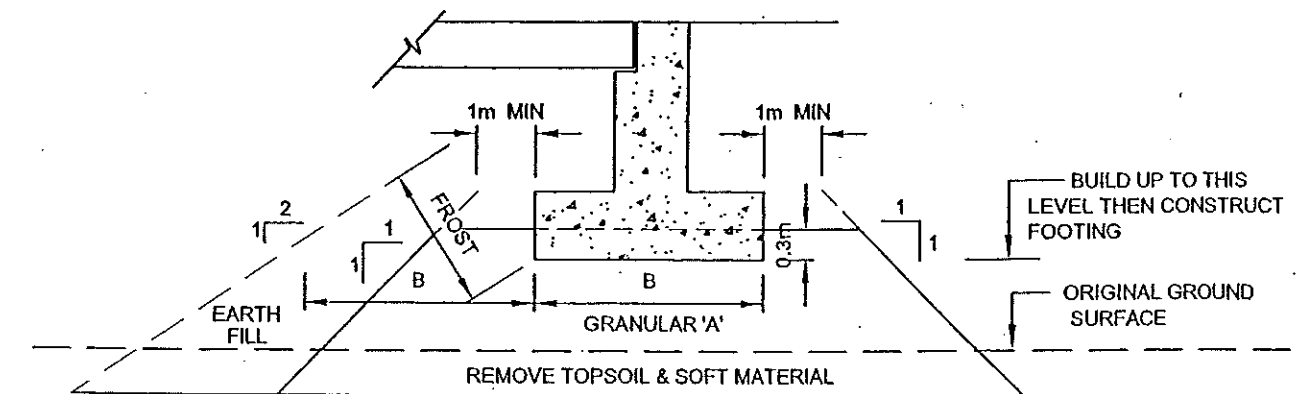
Earth Pressure Coefficients for Earthquake Loading (CHBDC 2000)  
PHA = .075g

Condition	Earth Pressure Coefficient (K) for Earthquake Loading					
	Granular A or Granular B-II		Granular B-I		Rock Fill	
	$\phi=35^\circ$	$\delta=17^\circ$	$\phi=32^\circ$	$\delta=16^\circ$	$\phi=42^\circ$	$\delta=21^\circ$
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
	Yielding Wall					
	Active (KAE)	0.27	0.42	0.30	0.51	0.20
	Passive (KPE)	6.98	63.19	5.60	34.55	14.27
	Non-Yielding Wall					
	Active (KAE)	0.31	0.58	0.35	-	0.24
	Passive (KPE)	6.56	61.15	5.25	33.38	13.49
						-

March, 2004



CROSS-SECTION




LONGITUDINAL SECTION

NOT TO SCALE

NOTES:

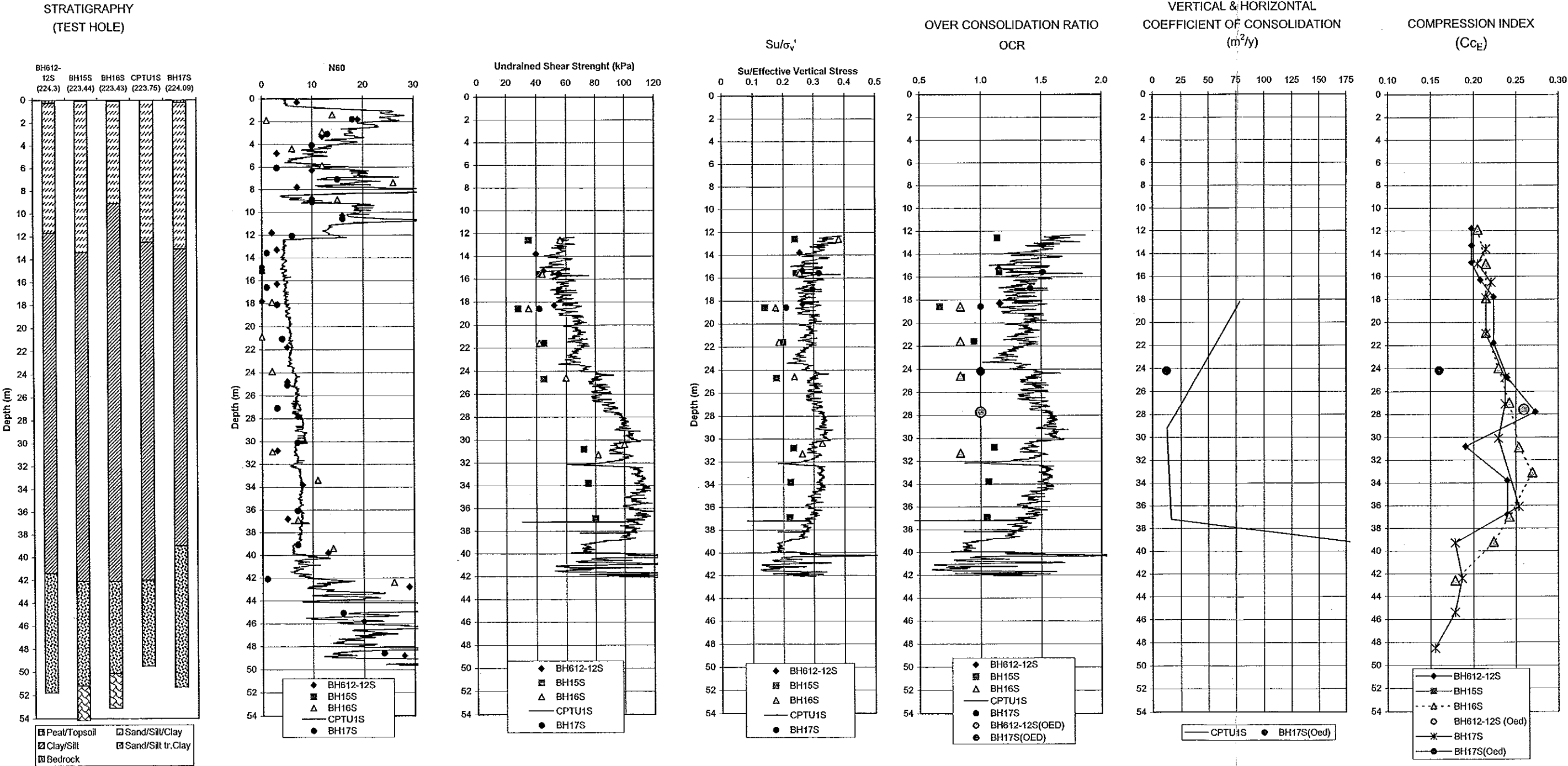
1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO O.P.S.S. 501.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. SOURCE M.T.C. 1982.

TED35146.DWG

ENGINEER	AEG	ABUTMENT ON COMPACTED FILL SHOWING GRANULAR A CORE	 <b>THURBER</b>
DRAWN	SS		
DATE	April, 2004		
APPROVED	PKC		
SCALE	NTS		
			DWG. NO. <b>FIGURE 7.1</b>



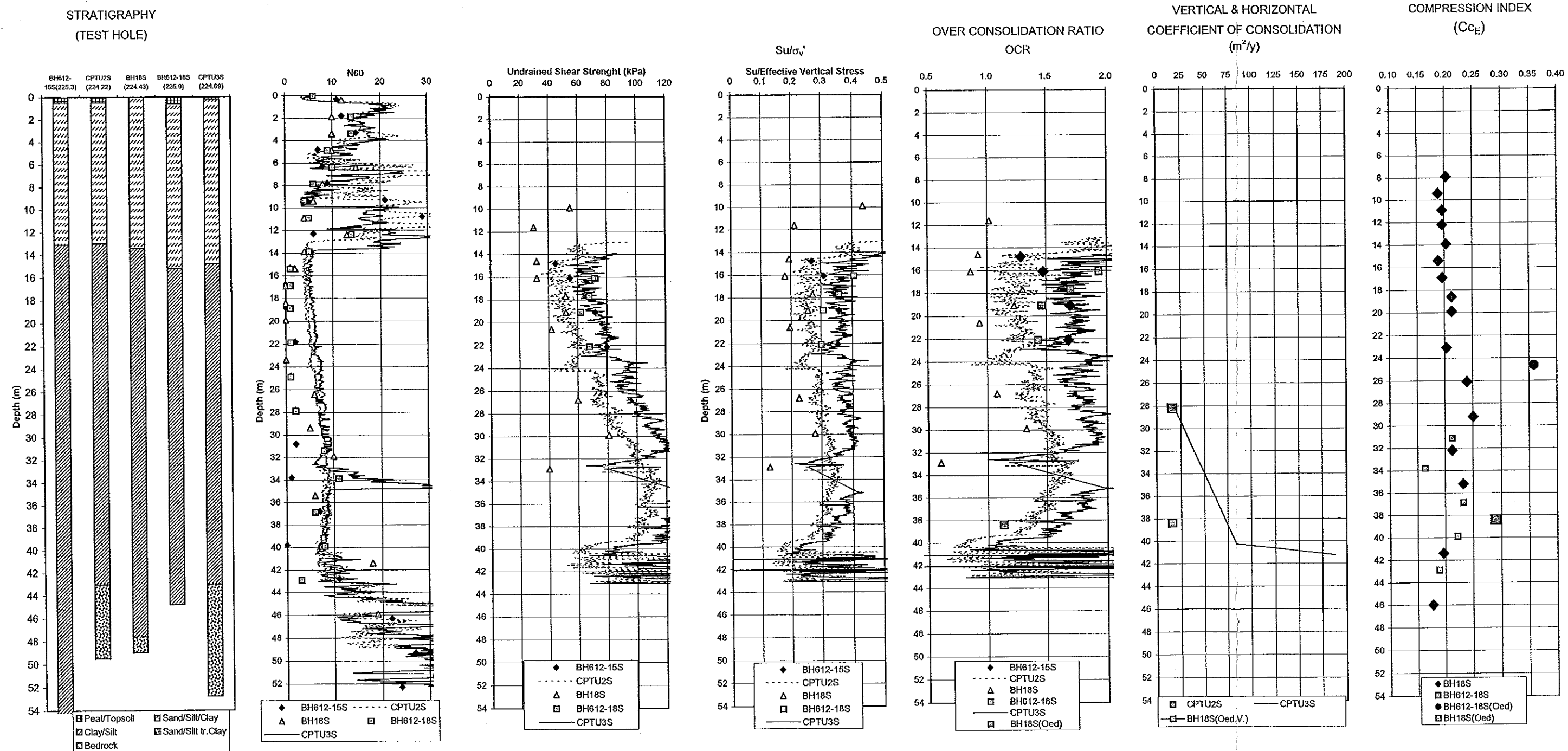
HIGHWAY 69 FOUR LANEING - ESTAIRE  
SUMMARY OF SUBSURFACE CONDITIONS  
SBL BRIDGE - NORTH ABUTMENT (10+550 to 10+600)



MASTER PLOT

FIGURE 10.1

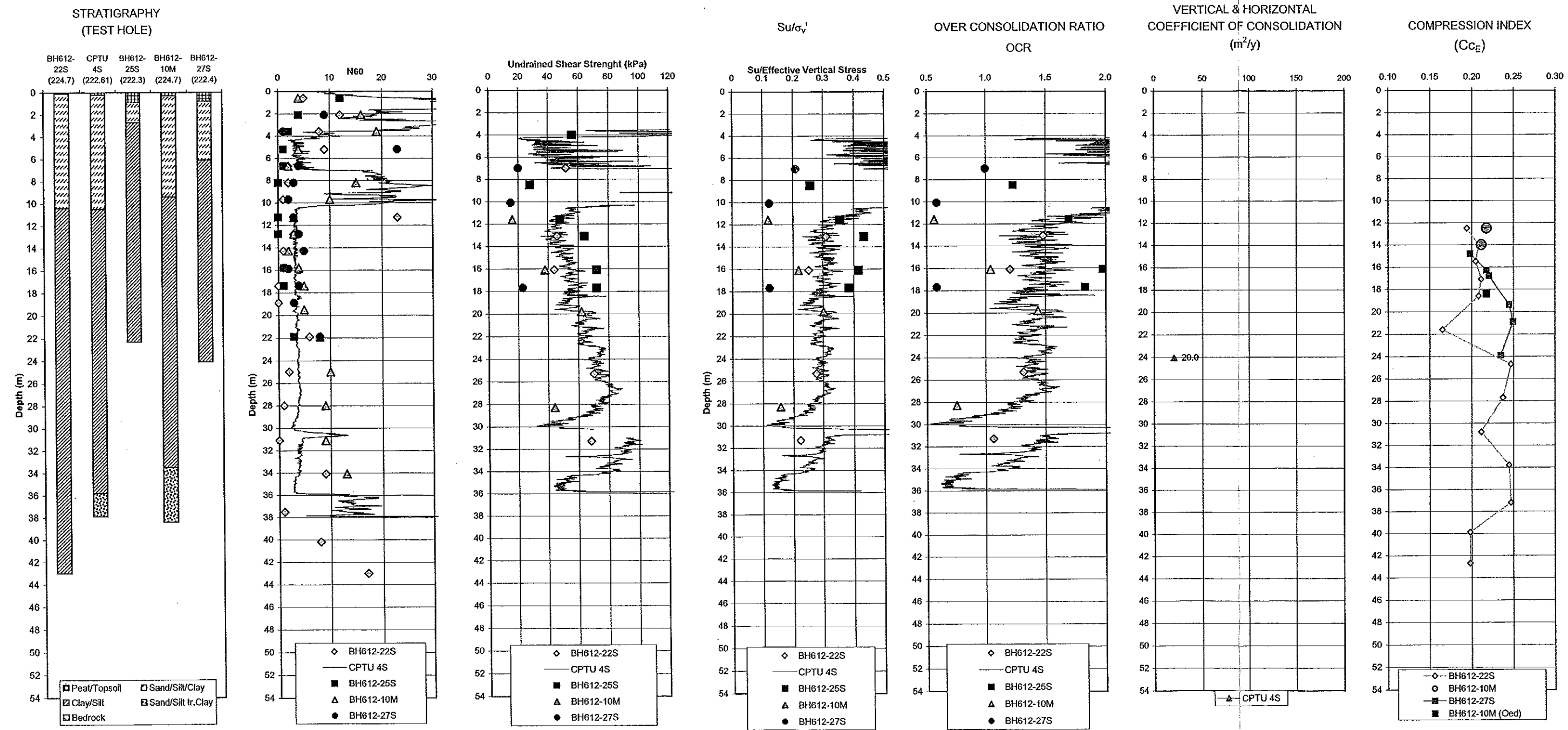
HIGHWAY 69 FOUR LANING - ESTAIRE  
SUMMARY OF SUBSURFACE CONDITIONS  
SBL BRIDGE - NORTH APPROACH-1 (10+600 to 10+675)



MASTER PLOT

FIGURE 10.2

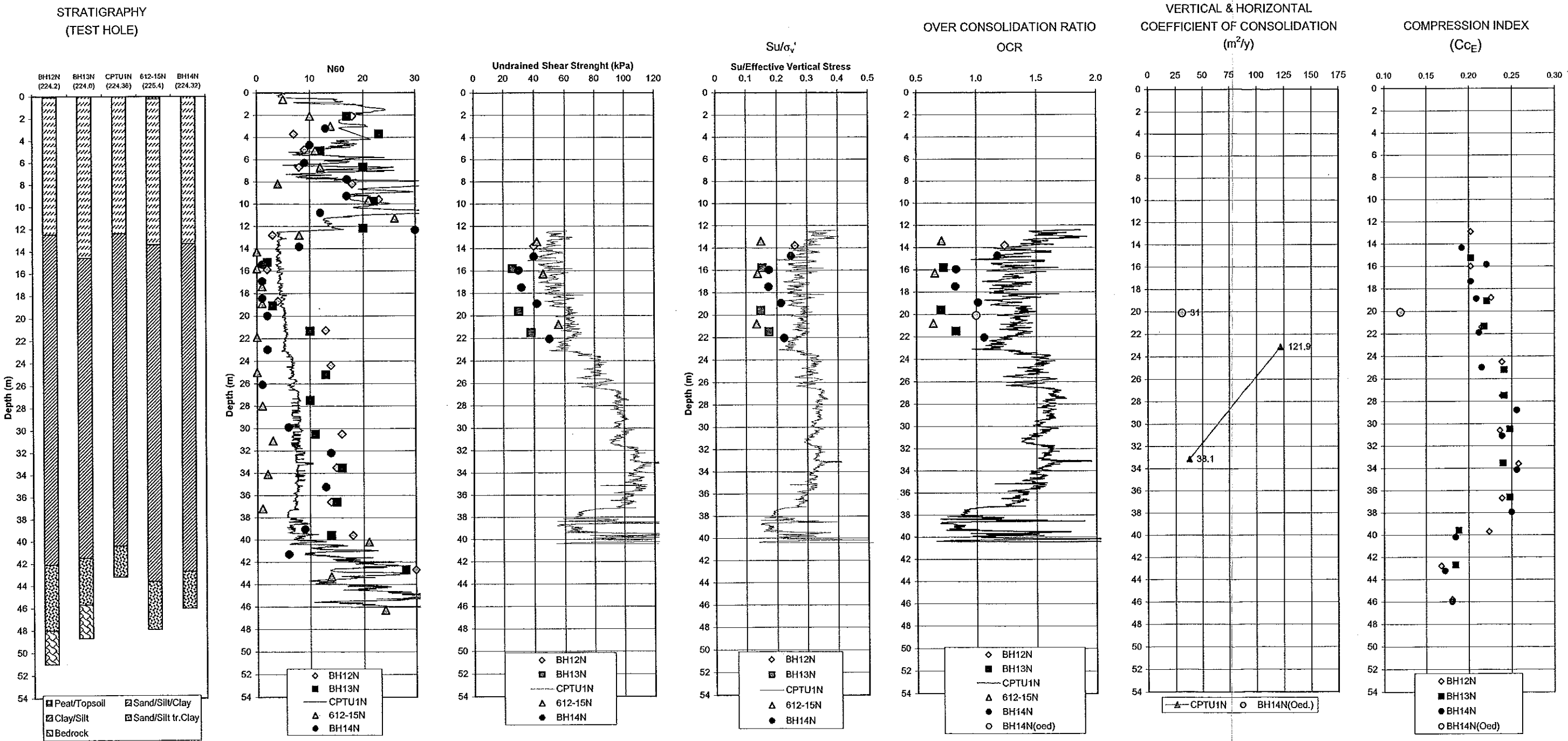
HIGHWAY 69 FOUR LANEING - ESTAIRE  
SUMMARY OF SUBSURFACE CONDITIONS  
SBL BRIDGE - NORTH APPROACH-2 (10+675 to Dill TWP 10+010)



MASTER PLOT

FIGURE 10.3

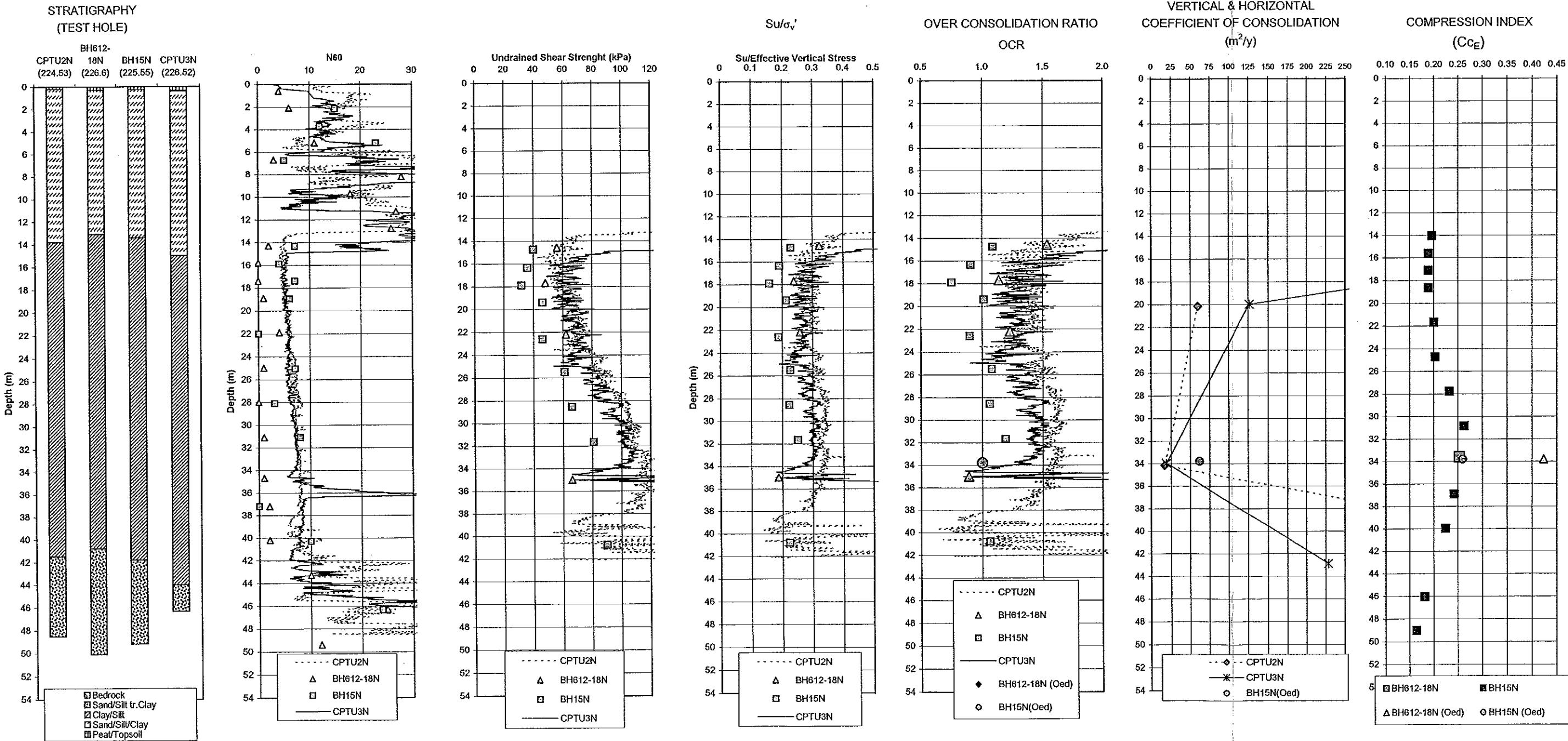
HIGHWAY 69 FOUR LANE - ESTAIRE  
SUMMARY OF SUBSURFACE CONDITIONS  
NBL BRIDGE - NORTH ABUTMENT (10+580 to 10+620)



MASTER PLOT

FIGURE 10.4

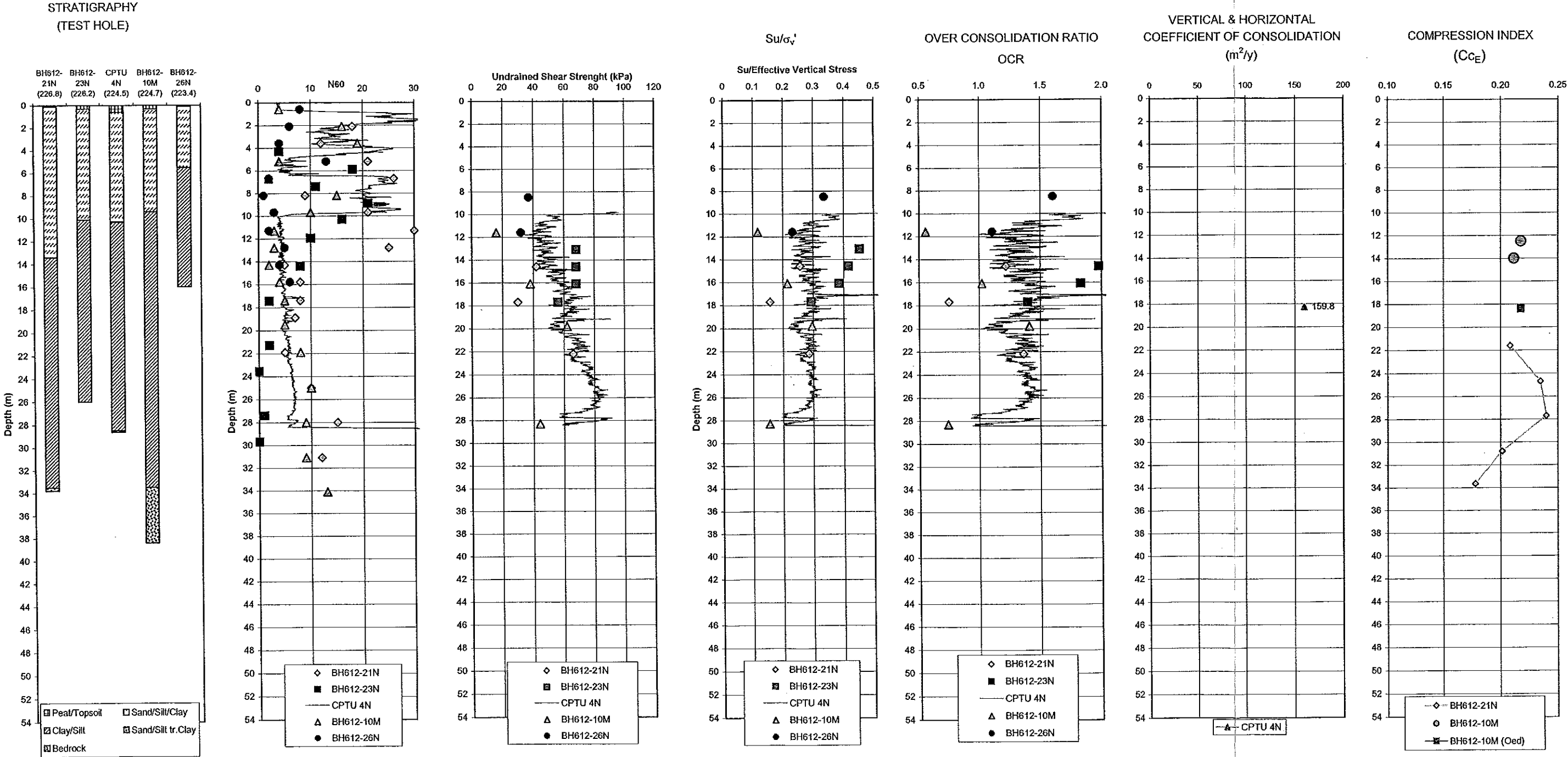
HIGHWAY 69 FOUR LANING - ESTAIRE  
SUMMARY OF SUBSURFACE CONDITIONS  
NBL BRIDGE - NORTH APPROACH-1 (10+620 to 10+675)



MASTER PLOT

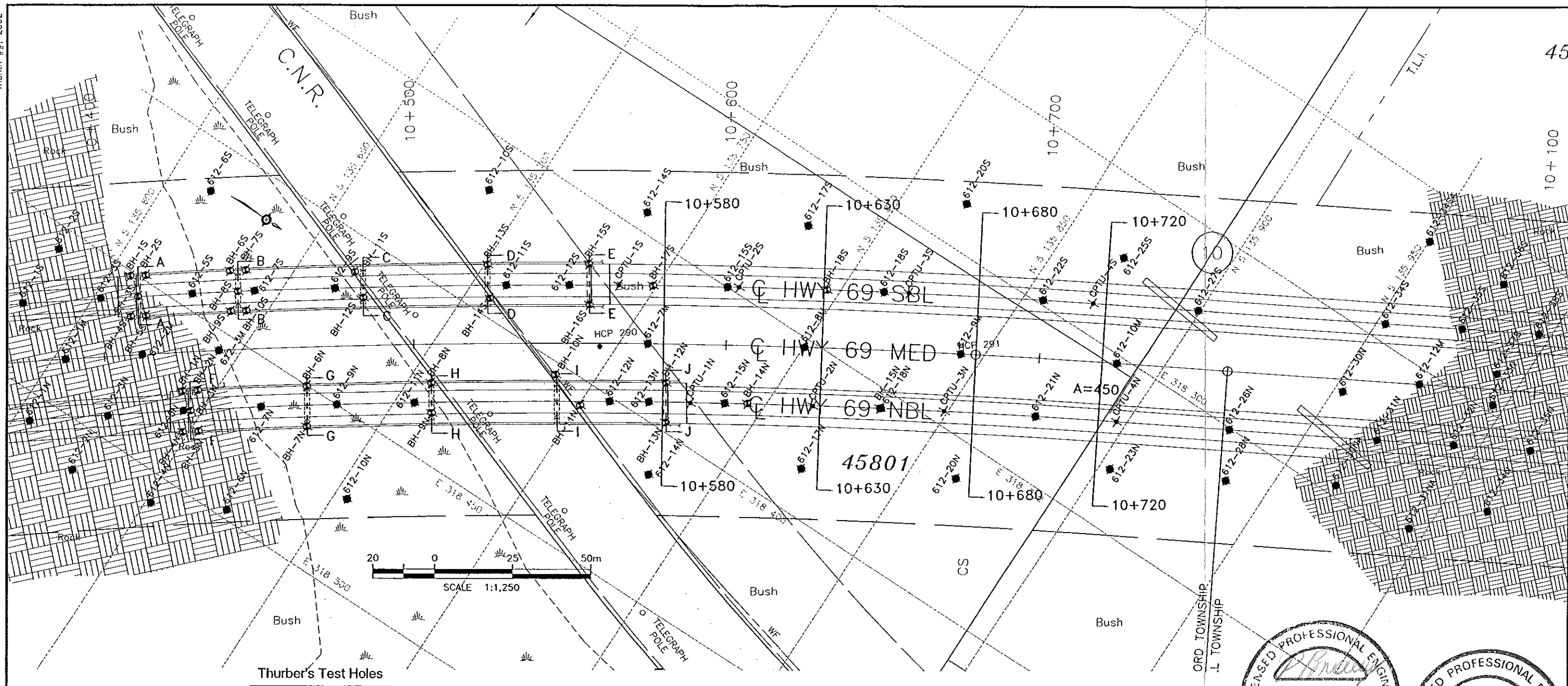
FIGURE 10.5

HIGHWAY 69 FOUR LANING - ESTAIRE  
SUMMARY OF SUBSURFACE CONDITIONS  
NBL BRIDGE - NORTH APPROACH-2 (10+675 to Dill TWP 10+010)



MASTER PLOT

FIGURE 10.6



NO	ELEVATION	NORTHING	EASTING
BH-1S	223.00	5135606.32	318451.37
BH-2S	221.69	5135610.49	318448.47
BH-3S	224.28	5135612.10	318455.54
BH-4S	225.48	5135613.57	318462.23
BH-5S	225.19	5135617.67	318459.32
BH-6S	218.99	5135632.76	318432.38
BH-7S	219.05	5135636.85	318429.49
BH-8S	219.09	5135638.49	318436.57
BH-9S	219.24	5135640.08	318443.14
BH-10S	219.20	5135644.11	318440.28
BH-11S	219.17	5135666.34	318410.91
BH-12S	219.27	5135673.16	318416.36
BH-13S	219.14	5135700.88	318385.65
BH-14S	219.03	5135707.44	318394.36
BH-15S	223.44	5135727.91	318367.90
BH-16S	223.43	5135735.20	318378.72
BH-17S	224.09	5135748.97	318362.46
BH-18S	224.43	5135796.14	318333.32
CPTU-1S	223.75	5135739.72	318368.36
CPTU-2S	224.22	5135772.18	318347.95
CPTU-3S	224.69	5135818.74	318319.86
CPTU-4S	222.61	5135869.65	318290.59

NO	ELEVATION	NORTHING	EASTING
BH-1N	224.12	5135641.05	318473.15
BH-2N	223.36	5135645.04	318470.24
BH-3N	223.62	5135646.67	318477.06
BH-4N	223.58	5135648.26	318484.12
BH-5N	223.32	5135652.40	318481.14
BH-6N	219.00	5135673.63	318450.12
BH-7N	219.24	5135680.77	318461.00
BH-8N	219.23	5135706.69	318427.42
BH-9N	219.16	5135712.04	318435.44
BH-10N	220.25	5135738.40	318403.54
BH-11N	220.96	5135750.44	318407.54
BH-12N	224.20	5135769.65	318386.48
BH-13N	224.00	5135776.52	318397.21
BH-14N	224.32	5135794.93	318377.88
BH-15N	225.55	5135831.26	318356.01
CPTU-1N	224.36	5135779.61	318387.67
CPTU-2N	224.53	5135812.90	318367.20
CPTU-3N	226.52	5135848.94	318345.56
CPTU-4N	224.50	5135896.95	318318.32

## LEGEND:

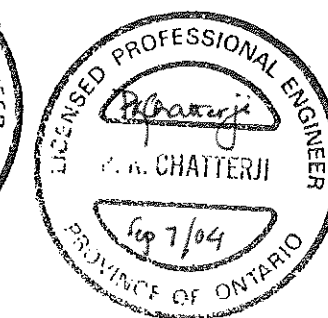
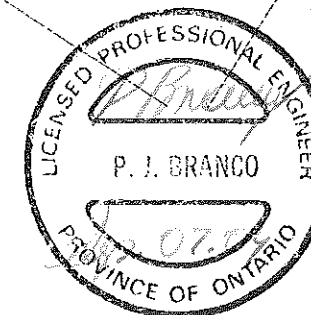
- ◆ Borehole Locations
- + CPTU Locations
- ◆ Boreholes by PetoMacCallum Ltd. ( Progress Report, May 2003 )

ENGINEER	PJB
DRAWN	SS
DATE	May 17, 2004
APPROVED	
SCALE	1: 1250

MINISTRY OF TRANSPORTATION ONTARIO

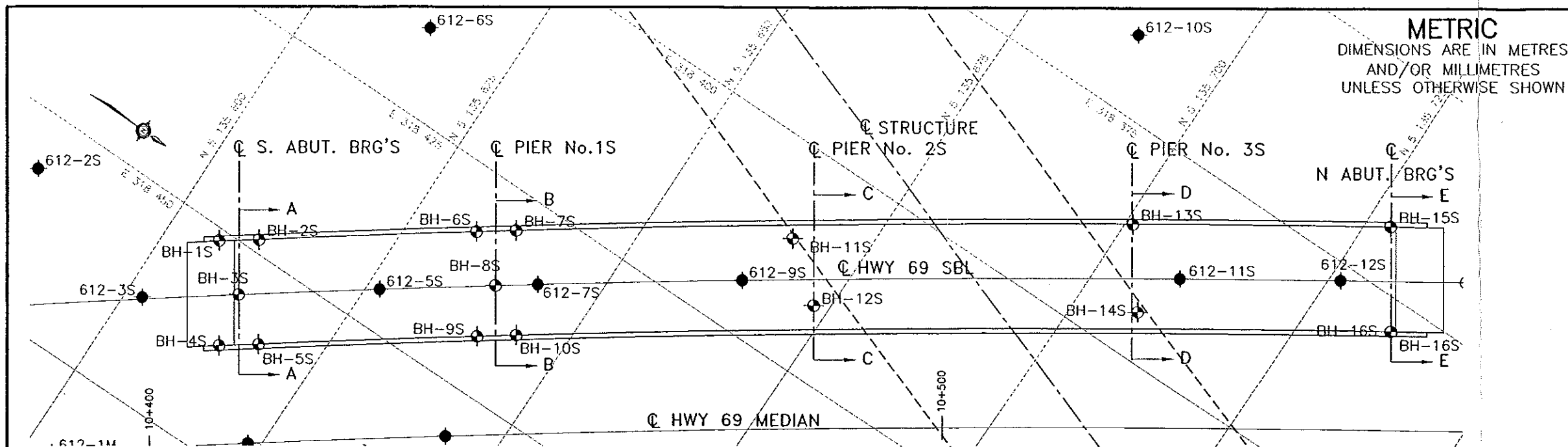
Geotechnical Investigation  
HIGHWAY 69 FOUR-LANING FOR 12km FROM 4km SOUTH  
OF ESTAIRE TO 1km NORTH OF HWY 537

TEST HOLE LOCATION PLAN

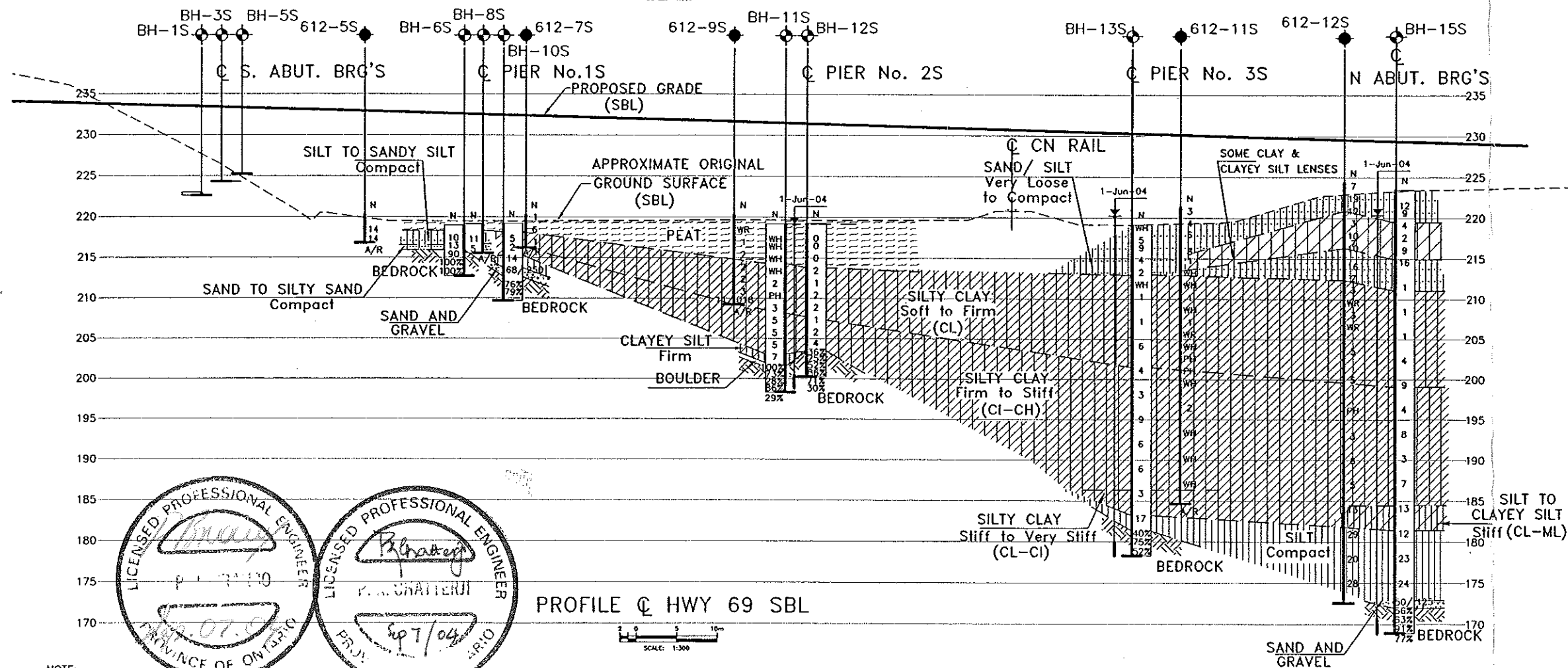


DRAWING NO.  
15-64-15-1



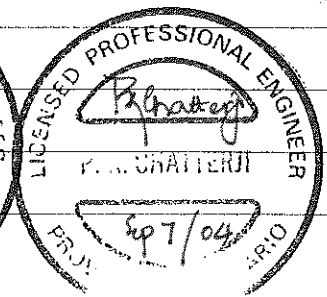
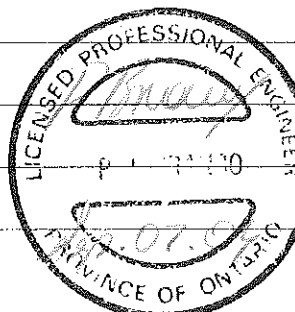


PLAN  
SCALE: 1:300



PROFILE  $\oslash$  HWY 69 SBL

SCALE: 1:300



NOTE:  
Information about boreholes drilled by PETO McALLUM LTD.  
were obtained from a report dated May 23, 2003

See Sheet 2 for cross-sections.

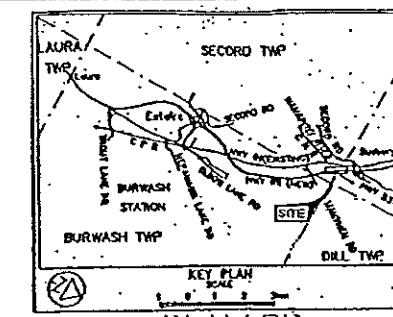
DRAWING NOT TO BE SCALED  
100mm ON ORIGINAL DRAWING

MILEAGE 245.67 BALA SUBDIVISION

DIST. 54  
CONT  
WP 5048-00-01  
HWY 69 FOUR-LANING-CNR  
OVERHEAD SBL STRUCTURE  
BOREHOLE LOCATIONS AND SOIL STRATA  
SHEET  
1



THURBER ENGINEERING LTD.  
THURBER



# LEGEND

- $\bullet$  Bore Hole by Peto MacCallum Ltd.
- $\oplus$  Piezocone by Conelec
- $\odot$  Bore Hole by Thurber
- N Blows/ 0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/ 0.3m (60° Cone, 475 J/blow)
- PH Pressure, Hydraulic
- WL at Time of Investigation
- Head Artesian Water
- Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
BH-1S	223.00	5135606.32	318451.37
BH-2S	221.69	5135610.49	318448.47
BH-3S	224.28	5135612.10	318455.54
BH-4S	225.48	5135613.57	318462.23
BH-5S	225.19	5135617.67	318459.32
BH-6S	218.99	5135632.76	318432.38
BH-7S	219.05	5135636.85	318429.49
BH-8S	219.09	5135638.49	318436.57
BH-9S	219.24	5135640.08	318443.14
BH-10S	219.20	5135644.11	318440.28
BH-11S	219.17	5135666.34	318410.91
BH-12S	219.27	5135673.16	318416.36
BH-13S	219.14	5135700.88	318385.65
BH-14S	219.03	5135707.44	318394.36
BH-15S	223.44	5135727.91	318367.90
BH-16S	223.43	5135735.20	318378.72

## NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS	DESCRIPTION
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DRAWN SS	CHK PJB SITE 46-494S DWG 1

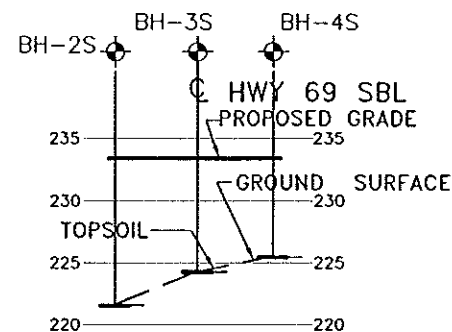
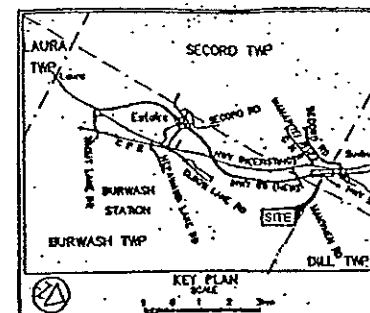
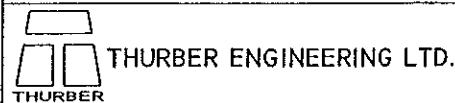


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DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

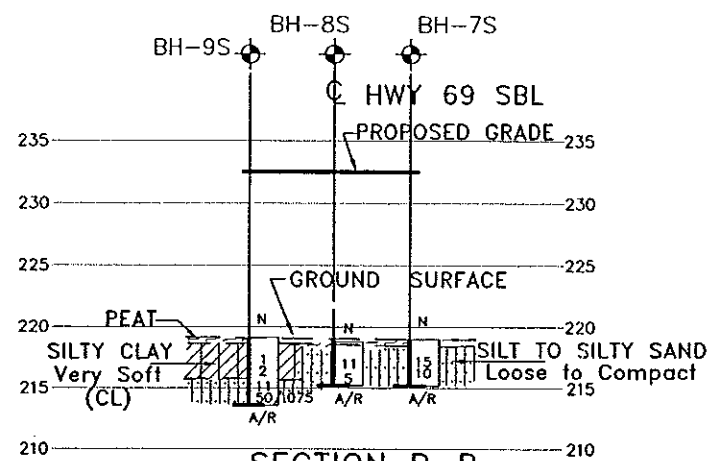
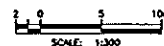
DIST. 54  
CONT  
WP 5048-00-01

HWY 69 FOUR-LANING-CNR  
OVERHEAD SBL STRUCTURE  
CROSS SECTIONS

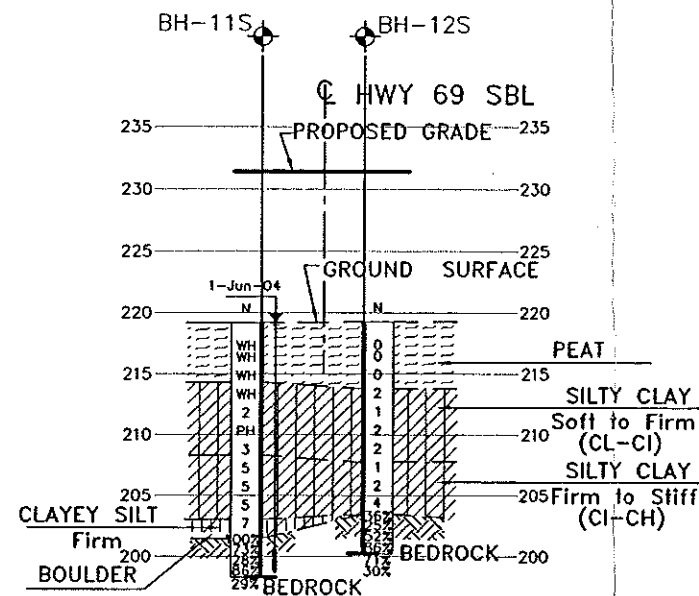
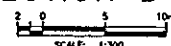
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2



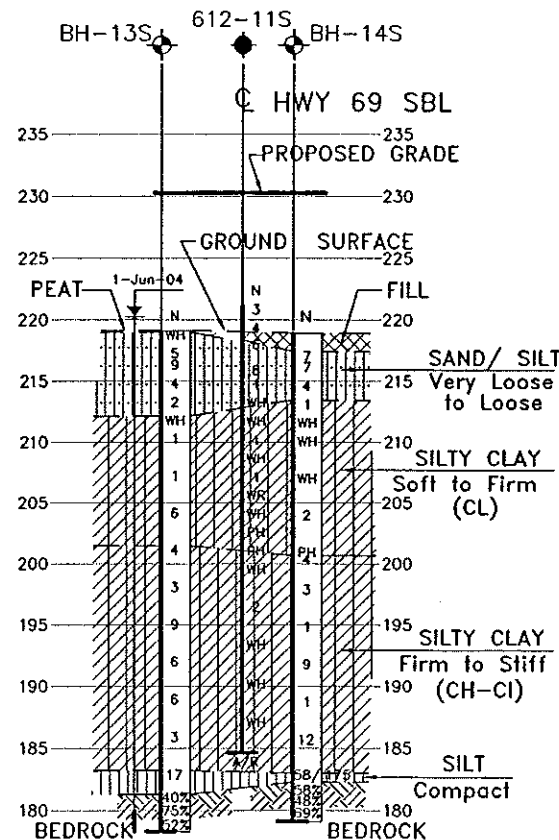
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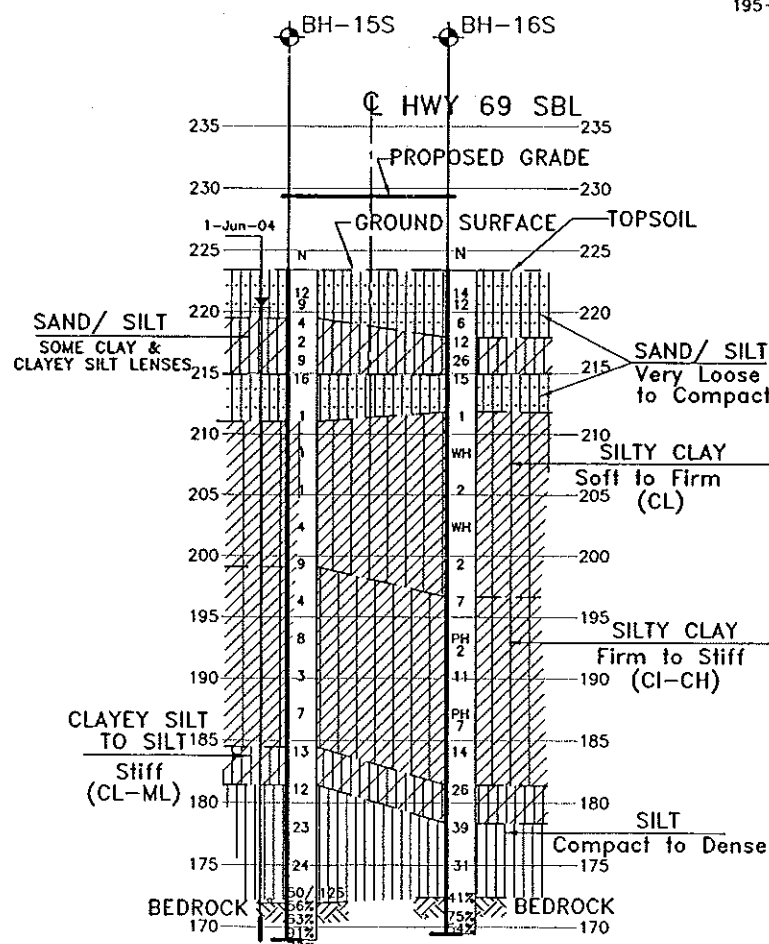
SECTION B-B



SECTION C-C



SECTION D-D



SECTION E-E



NOTE:  
Information about boreholes drilled by PETO McCALLUM LTD.  
were obtained from a report dated May 23, 2003

See Sheet 1 for section locations.

DRAWING NOT TO BE SCALED  
100mm ON ORIGINAL DRAWING

MILEAGE 245.67 BALA SUBDIVISION

DESIGN PJB CHK CODE CHBDC-00/LOAD CL625-ONT DATE JULY, 04  
DRAWN SS CHK PJB SITE 46-494S DWG 2

# LEGEND

- Bore Hole by Peto MacCallum Ltd.
- ⊕ Piezocone by Conelec
- ⊙ Bore Hole by Thurber
- N Blows/ 0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/ 0.3m (60' Cone, 475 J/blow)
- PH Pressure, Hydraulic
- WL at Time of Investigation
- Head Artesian Water
- Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
BH-1S	223.00	5135606.32	318451.37
BH-2S	221.69	5135610.49	318448.47
BH-3S	224.28	5135612.10	318455.54
BH-4S	225.48	5135613.57	318462.23
BH-5S	225.19	5135617.67	318459.32
BH-6S	218.99	5135632.76	318432.38
BH-7S	219.05	5135636.85	318429.49
BH-8S	219.09	5135638.49	318436.57
BH-9S	219.24	5135640.08	318443.14
BH-10S	219.20	5135644.11	318440.28
BH-11S	219.17	5135666.34	318410.91
BH-12S	219.27	5135673.16	318416.36
BH-13S	219.14	5135700.88	318385.65
BH-14S	219.03	5135707.44	318394.36
BH-15S	223.44	5135727.91	318367.90
BH-16S	223.43	5135735.20	318378.72

## NOTE

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DIST. 54  
CONT  
WP 5047-00-01



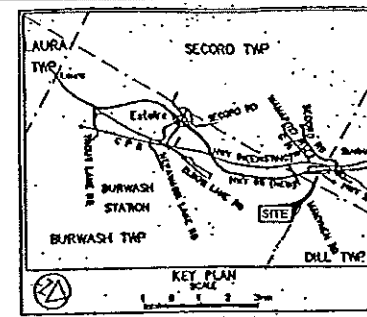
HWY 69 FOUR-LANING-CNR  
OVERHEAD NBL STRUCTURE  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET  
3



300 Water Street  
Windsor, Ontario  
N9A 5J2  
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Totten Stone Hubbert Associates (1997) Limited

THURBER ENGINEERING LTD.  
THURBER



### LEGEND

- Bore Hole by Peto MacCallum Ltd.
- ⊕ Piezocone by Conetec
- ⊙ Bore Hole by Thurber
- N Blows/ 0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/ 0.3m (60° Cone, 475 J/blow)
- PH Pressure, Hydraulic
- WL at Time of Investigation
- Head Artesian Water
- Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
BH-1N	224.12	5135641.05	318473.15
BH-2N	223.36	5135645.04	318470.24
BH-3N	223.62	5135646.67	318477.06
BH-4N	223.58	5135648.26	318484.12
BH-5N	223.32	5135652.40	318481.14
BH-6N	219.00	5135673.63	318450.12
BH-7N	219.24	5135680.77	318461.00
BH-8N	219.23	5135706.69	318427.42
BH-9N	219.16	5135712.04	318435.44
BH-10N	220.25	5135738.40	318403.54
BH-11N	220.96	5135750.44	318407.54
BH-12N	224.20	5135769.65	318386.48
BH-13N	224.00	5135776.52	318397.21

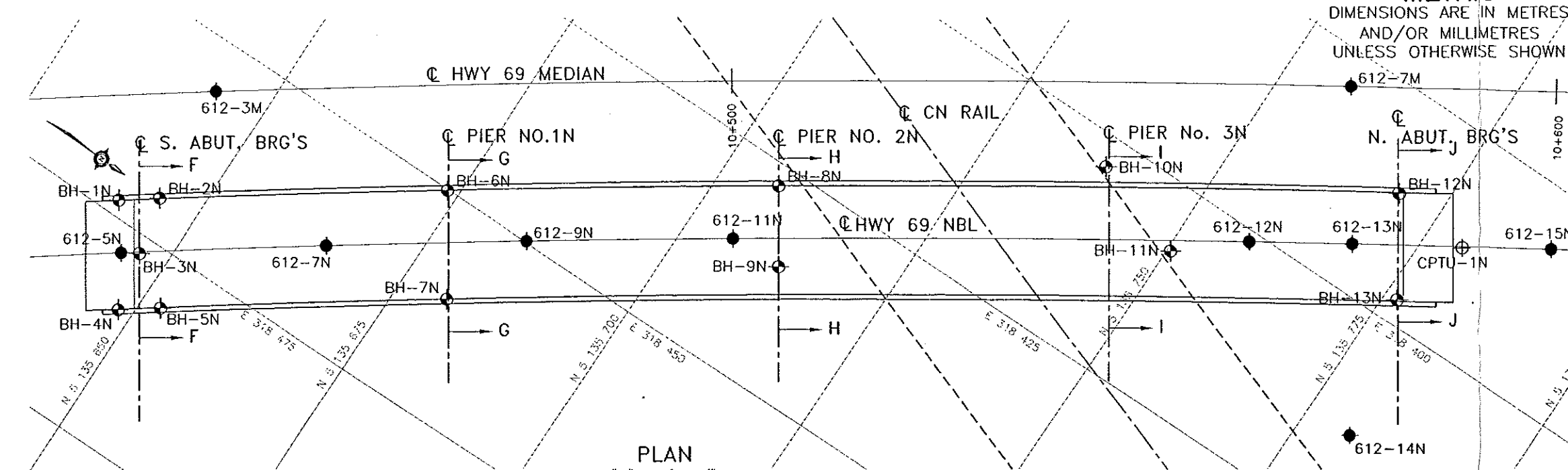
### NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS	DESCRIPTION
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DRAWN SS	CHK PJB
DATE JULY, 04	DWG 3

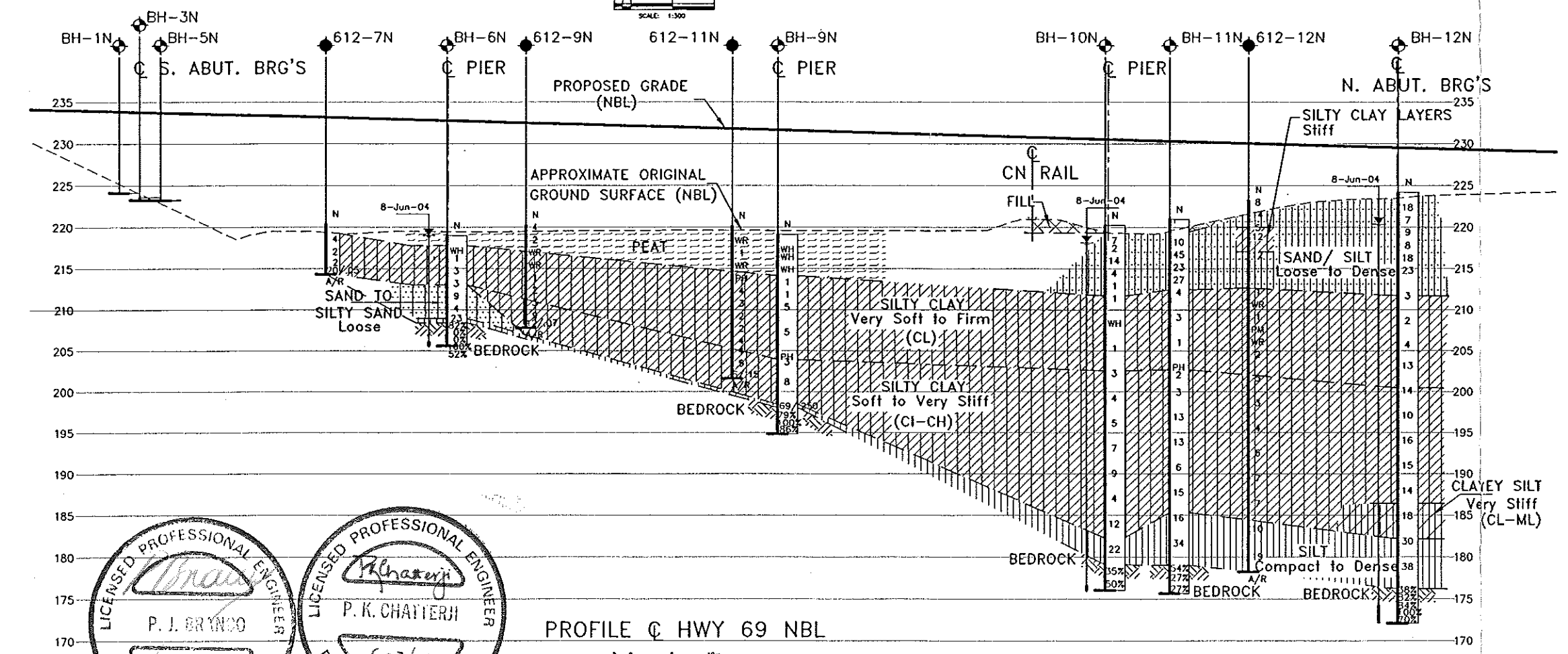
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METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN



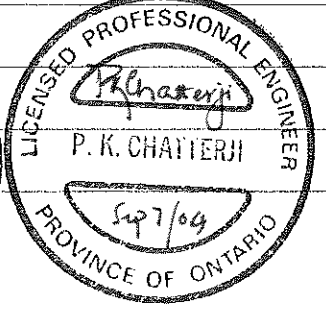
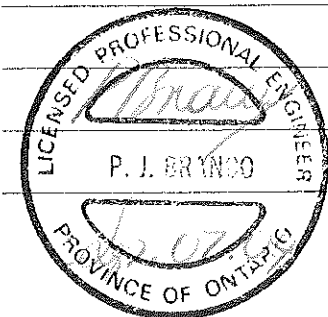
### PLAN

SCALE: 1:300



### PROFILE & HWY 69 NBL

SCALE: 1:300



NOTE:  
Information about boreholes drilled by PETO McCALLUM LTD.  
were obtained from a report dated May 23, 2003  
See Sheet 4 for cross-sections

DRAWING NOT TO BE SCALED  
100mm ON ORIGINAL DRAWING

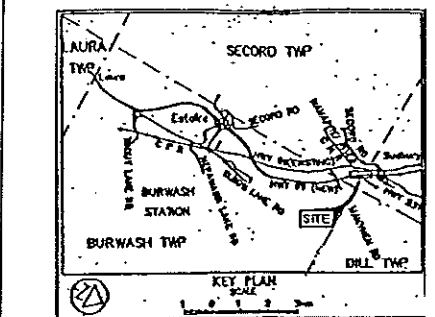
MILEAGE 245.67 BALA SUBDIVISION

**METRIC**  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

DIST. 54  
CONT  
WP 5047-00-01

HWY 69 FOUR-LANING-CNR  
OVERHEAD NBL STRUCTURE  
CROSS SECTIONS

SHEET  
4



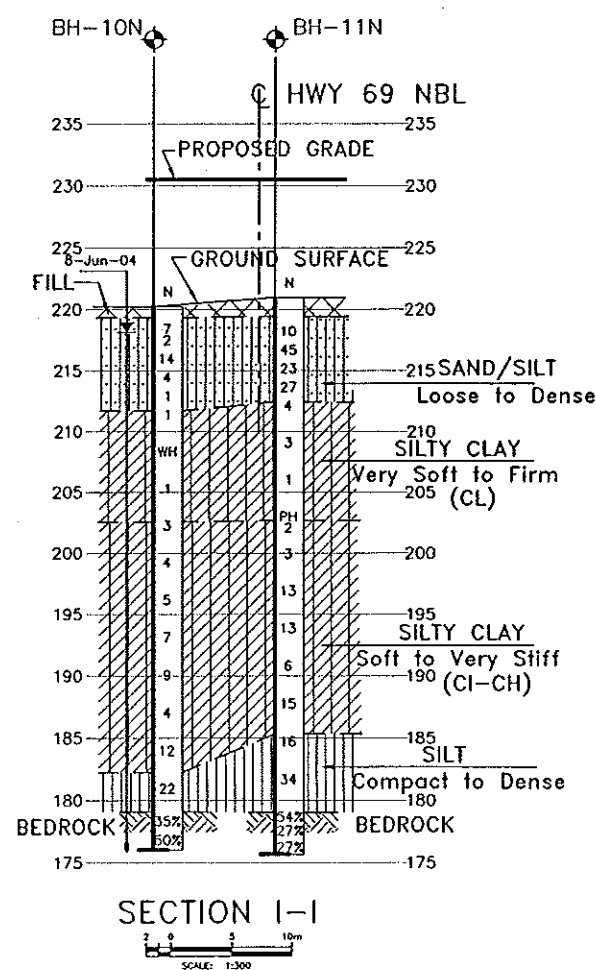
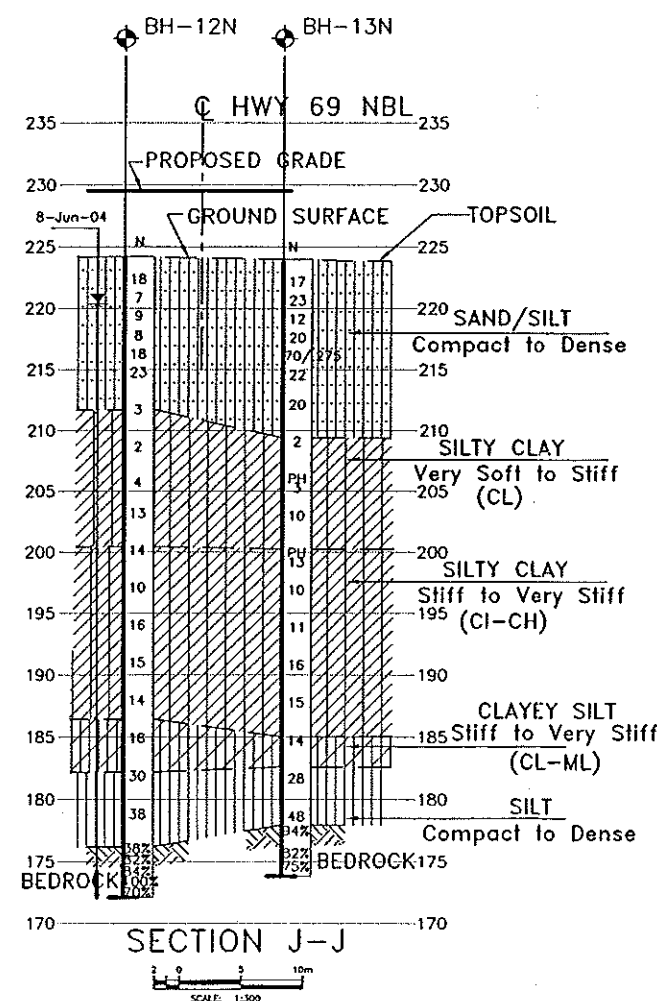
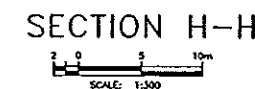
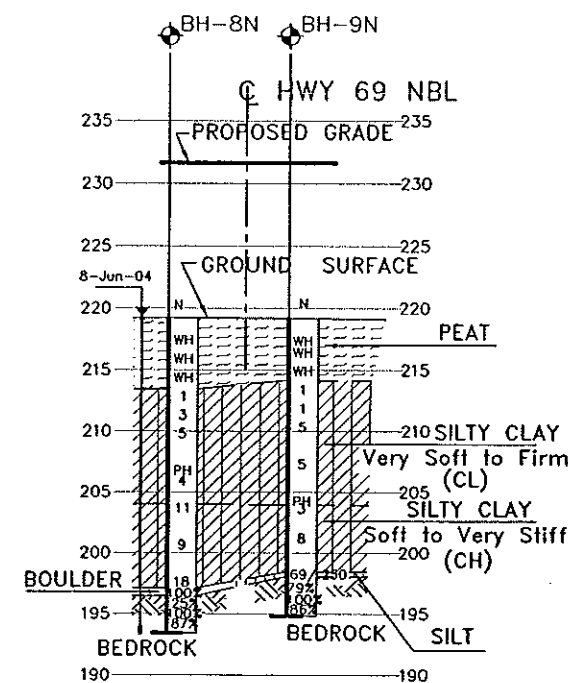
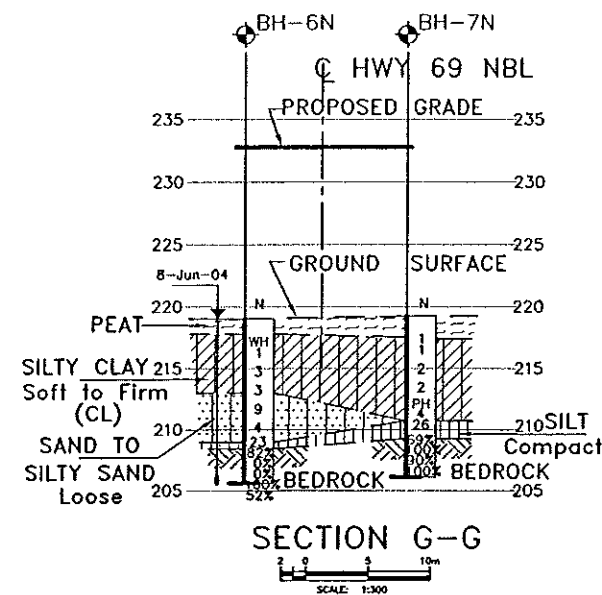
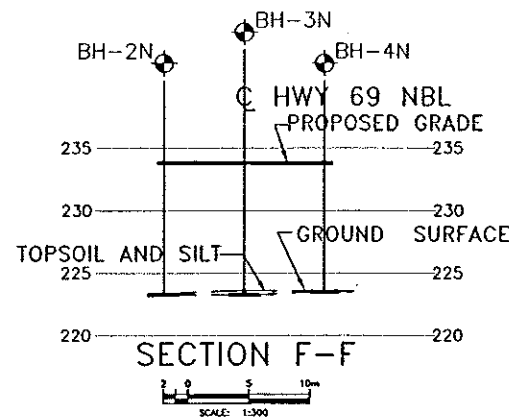
# LEGEND

- Bore Hole by Peto MacCollum Ltd.
- ⊕ Piezocone by Conetec
- ⊙ Bore Hole by Thurber
- N Blows/ 0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/ 0.3m (60' Cone, 475 J/blow)
- PH Pressure, Hydraulic
- WL at Time of Investigation
- ↑ Head Artesian Water
- Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
BH-1N	224.12	5135641.05	318473.15
BH-2N	223.36	5135645.04	318470.24
BH-3N	223.62	5135646.67	318477.06
BH-4N	223.58	5135648.26	318484.12
BH-5N	223.32	5135652.40	318481.14
BH-6N	219.00	5135673.63	318450.12
BH-7N	219.24	5135680.77	318461.00
BH-8N	219.23	5135706.69	318427.42
BH-9N	219.16	5135712.04	318435.44
BH-10N	220.25	5135738.40	318403.54
BH-11N	220.96	5135750.44	318407.54
BH-12N	224.20	5135769.65	318386.48
BH-13N	224.00	5135776.52	318397.21

## NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.



NOTE:  
Information about boreholes drilled by PETO McCALLUM LTD.  
were obtained from a report dated May 23, 2003  
See Sheet 3 for section locations.

DRAWING NOT TO BE SCALED  
100mm ON ORIGINAL DRAWING

MILEAGE 245.67 BALA SUBDIVISION

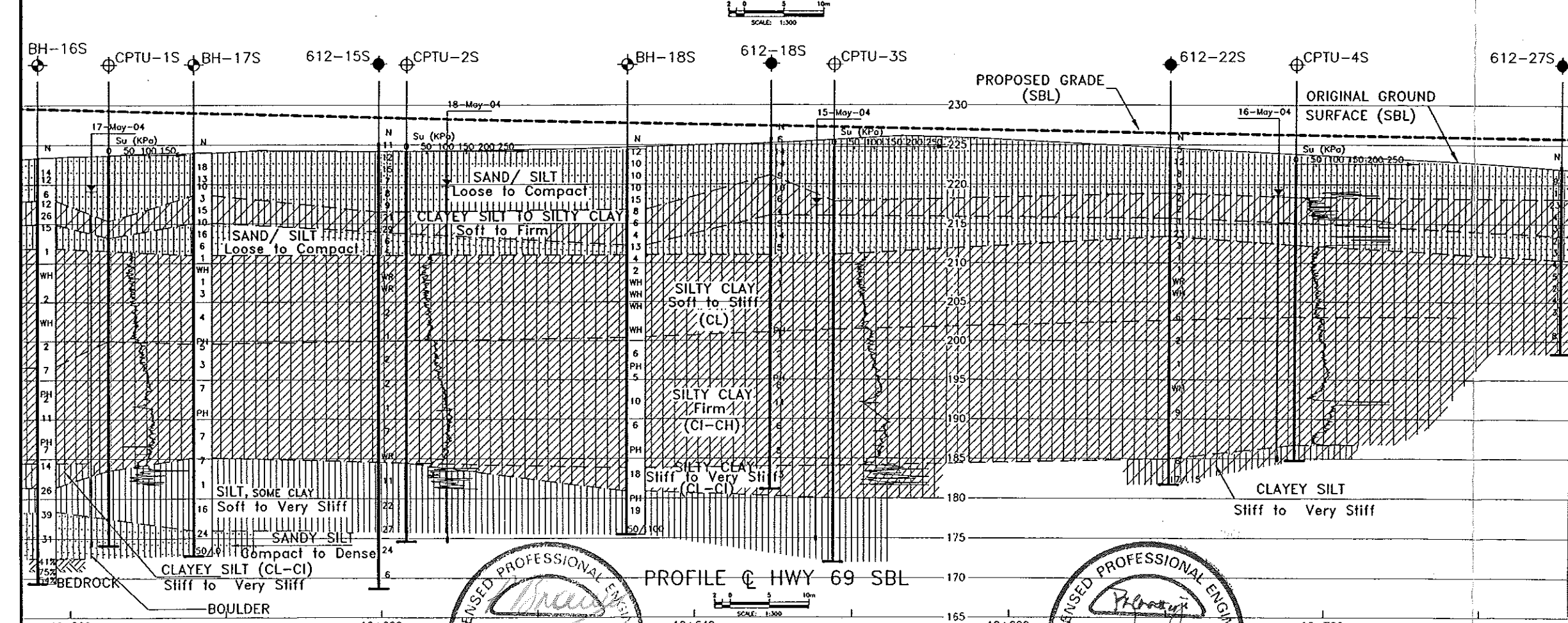
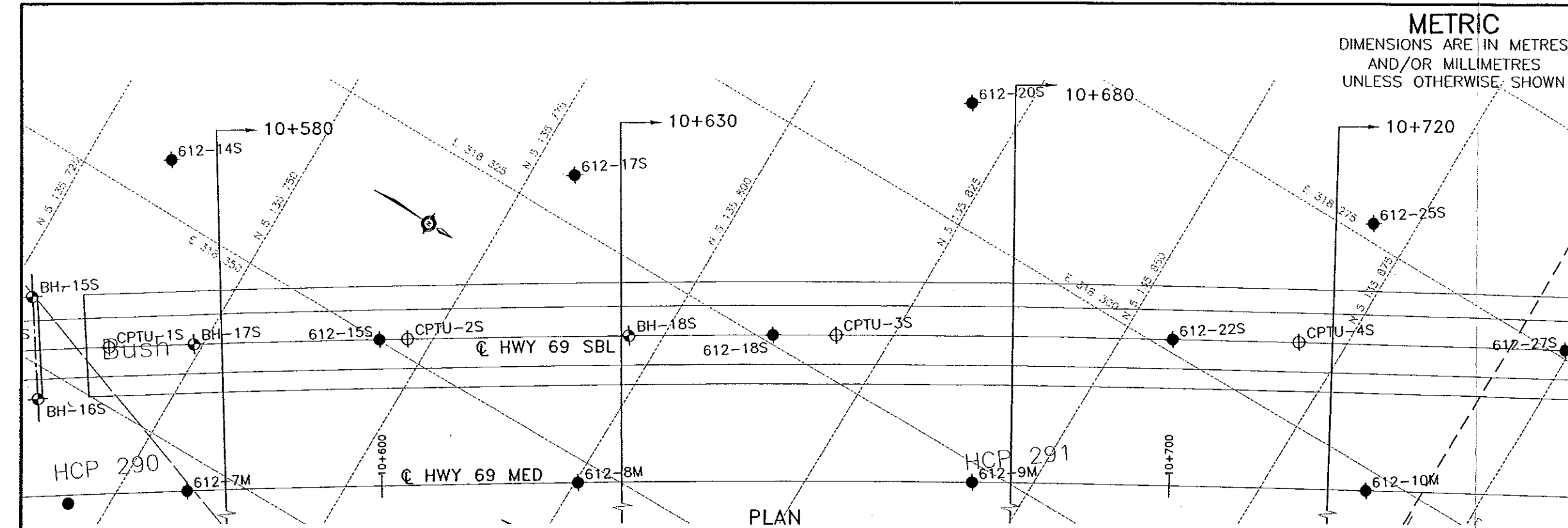
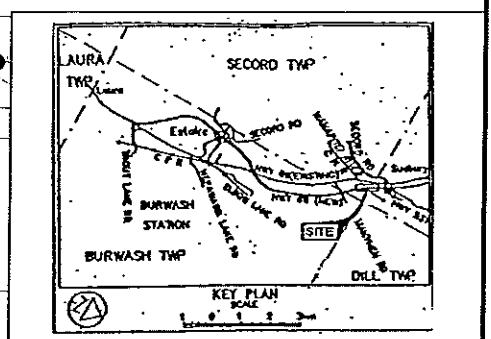
DESIGN PJB CHK CODE CHBDC-00 LOAD CL625-ONT DATE JULY, 04  
DRAWN SS CHK PJB SITE 46-494N DWG 4

METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

DIST. 54  
CONT  
WP 5048-00-01  
HWY 69 FOUR-LANING-CNR OVERHEAD SBL  
STRUCTURE- NORTH APPROACH EMBANKMENT  
BOREHOLE LOCATIONS AND SOIL STRATA  
SHEET  
5

**SH**  
engineers  
architects  
planners  
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L9N 5P2  
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www.sh.ca  
Totten Sims Hubicki Associates (1997) Limited

**THURBER**  
THURBER ENGINEERING LTD.



LEGEND			
●	Bore Hole by Peto MacCallum Ltd.		
⊕	Piezometer by Conelec (CPTU)		
⊙	Bore Hole by Thurber		
N	Blows/ 0.3m (Std Pen Test, 475 J/blow)		
CONE	Blows/ 0.3m (60' Cone, 475 J/blow)		
PH	Pressure, Hydraulic		
WL	WL at Time of Investigation		
↑	Head Artesian Water		
⊕	Piezometer		
90%	Rock Quality Designation (RQD)		
A/R	Auger Refusal		
NO	ELEVATION	NORTHING	EASTING
BH-15S	223.44	5135727.91	318367.90
BH-16S	223.43	5135735.20	318378.72
BH-17S	224.09	5135748.97	318362.46
BH-18S	224.43	5135796.14	318333.32
CPTU-1S	223.75	5135739.72	318368.36
CPTU-2S	224.22	5135772.18	318347.95
CPTU-3S	224.69	5135818.74	318319.86
CPTU-4S	222.61	5135869.65	318290.59

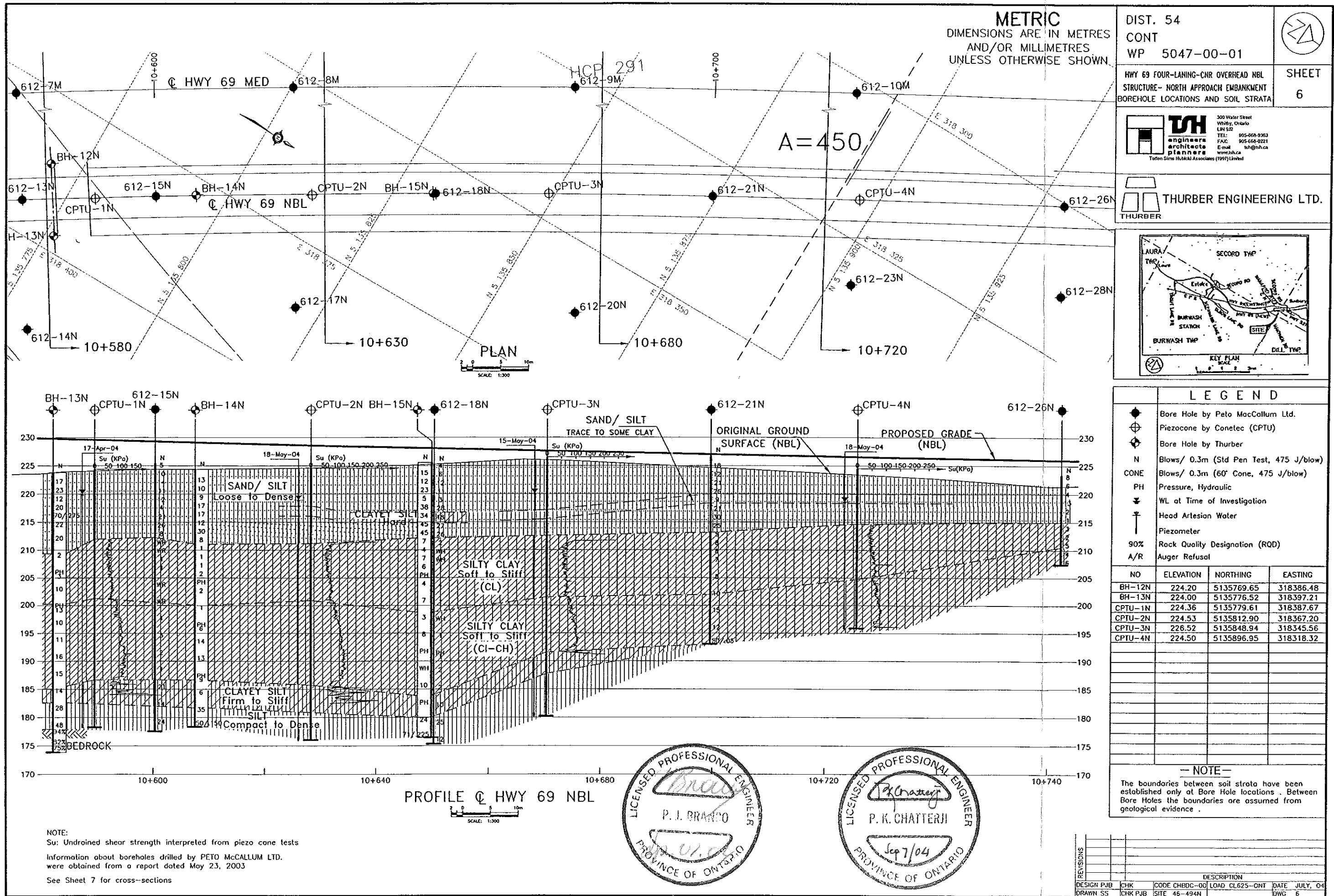
NOTE:  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE:  
Su: Undrained shear strength interpreted from piezo cone tests  
Information about boreholes drilled by PETO McCALLUM LTD. were obtained from a report dated May 23, 2003  
See Sheet 7 for cross-sections

LICENSED PROFESSIONAL ENGINEER  
P. J. BR...  
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER  
P. K. CHATTERJI  
PROVINCE OF ONTARIO

REVISIONS	DESCRIPTION
DESIGN PJB	CHK CODE CHBDC-00 LOAD CL625-ONT DATE JULY, 04
DRAWN SS	CHK PJB SITE 46-4945 DWG 5



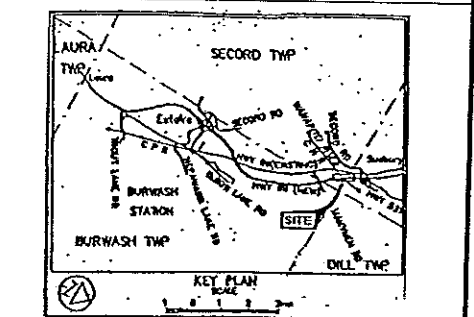


METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

DIST. 54  
CONT 5047-00-01  
WP 5048-00-01  
HWY 69 FOUR-LANING-CHN OVERHEAD  
STRUCTURES- NORTH APPROACH EMBANKMENTS  
CROSS SECTIONS  
SHEET 7

**SH**  
engineers  
architects  
planners  
Totten Sims Hubert Associates (1997) Limited  
300 Water Street  
Whitby, Ontario  
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TEL: 905-668-9363  
FAX: 905-668-0221  
E-mail: sh@sh.ca  
www.sh.ca

**THURBER**  
THURBER ENGINEERING LTD.



LEGEND

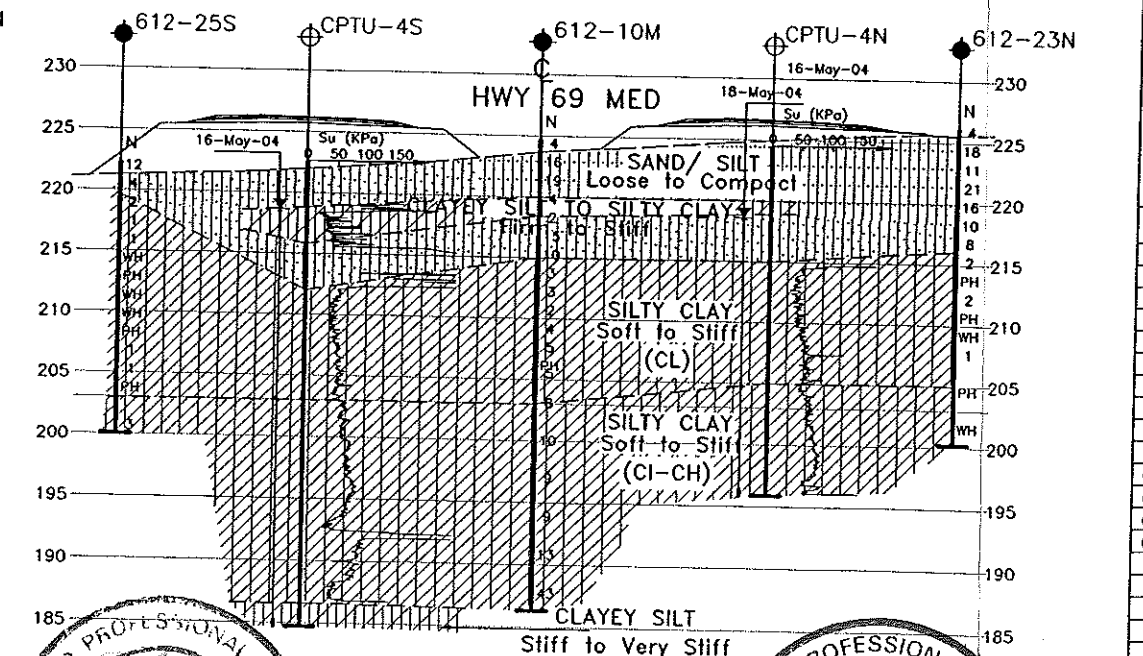
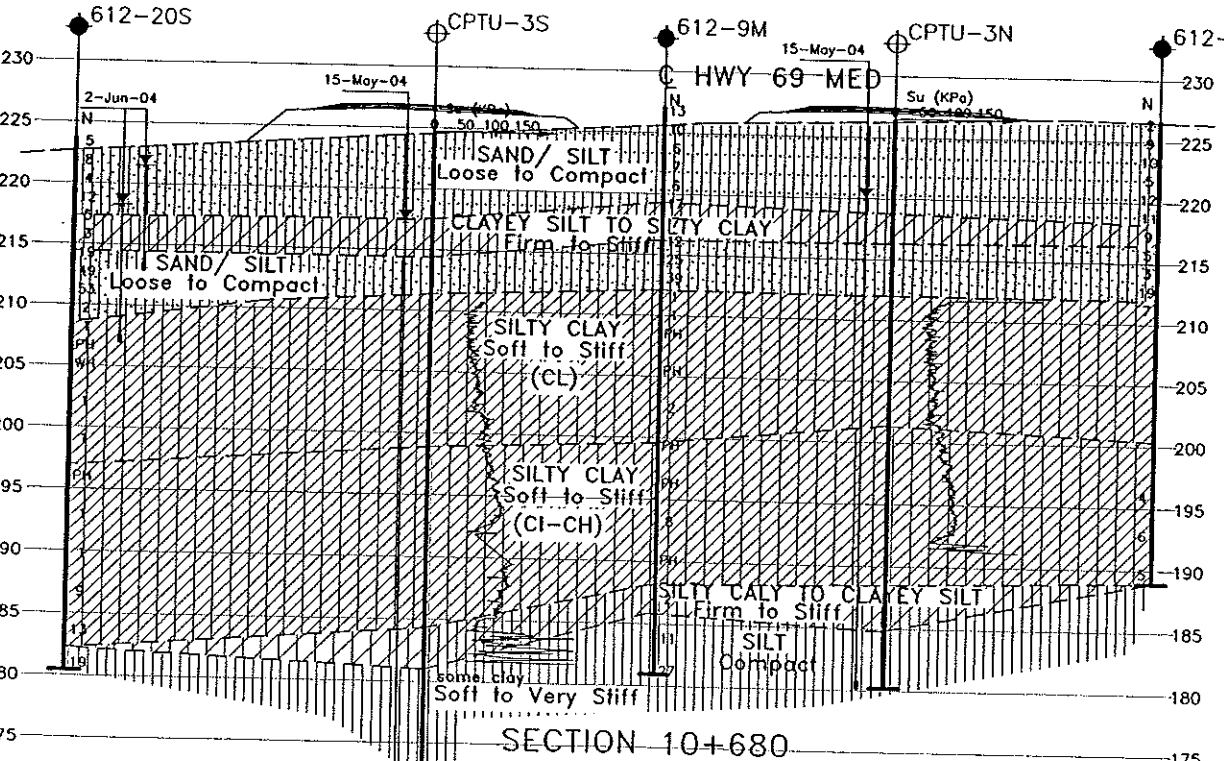
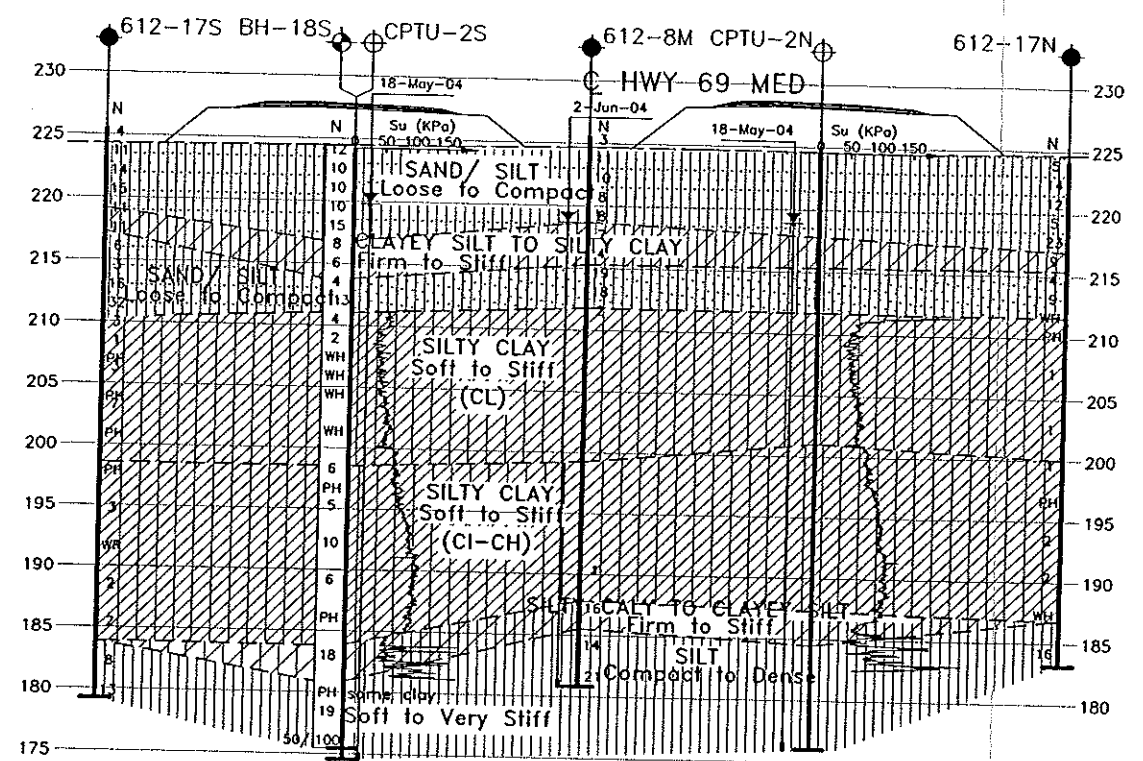
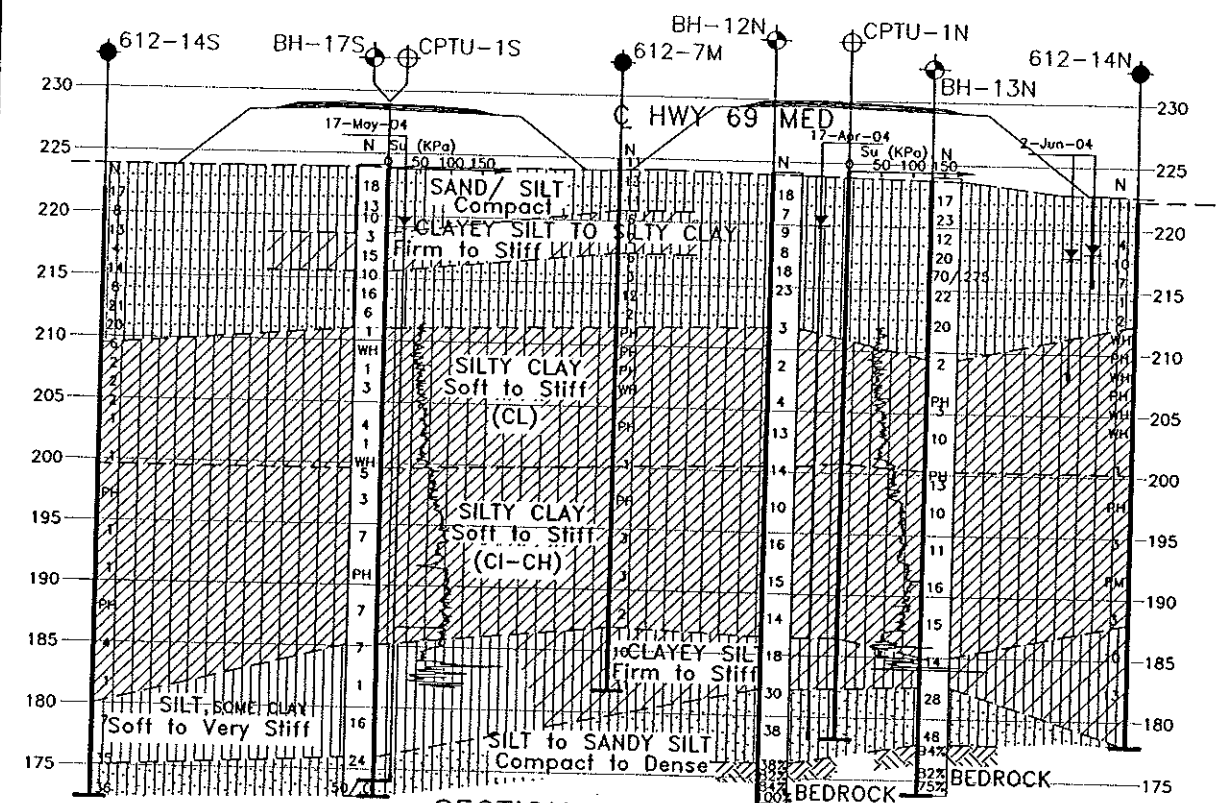
- Bore Hole by Peto MacCallum Ltd.
- ⊕ Piezocone by Conelec (CPTU)
- ⊙ Bore Hole by Thurber
- N Blows/ 0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/ 0.3m (60° Cone, 475 J/blow)
- PH Pressure, Hydraulic
- WL at Time of Investigation
- ↑ Head Artesian Water
- ⊕ Piezometer
- 90% Rock Quality Designation (ROD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
BH-12N	224.20	5135769.65	318386.48
BH-13N	224.00	5135776.52	318397.21
CPTU-1N	224.36	5135779.91	318387.67
CPTU-2N	224.53	5135812.90	318367.20
CPTU-3N	226.52	5135848.94	318345.56
CPTU-4N	224.50	5135896.95	318318.32
BH-15S	223.44	5135727.91	318367.90
BH-16S	223.43	5135735.20	318378.72
BH-17S	224.09	5135748.97	318362.46
BH-18S	224.43	5135796.14	318333.32
CPTU-1S	223.75	5135739.72	318368.36
CPTU-2S	224.22	5135772.18	318347.95
CPTU-3S	224.69	5135818.74	318319.86
CPTU-4S	222.61	5135869.65	318290.59

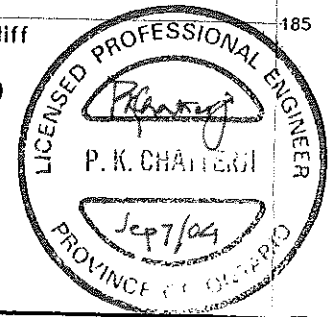
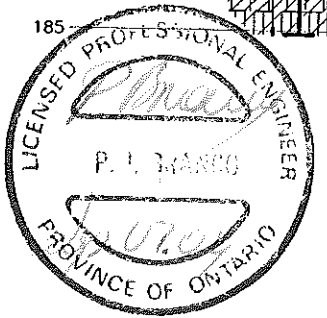
NOTE:  
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REVISIONS

NO	DESCRIPTION
1	DESIGN PJB
2	CHK PJB
3	CODE CHBCC-00/LOAD CL625-ONT
4	DATE JULY, 04
5	ORAWN SS
6	CHK PJB
7	SITE 46-494N/S
8	DATE JULY, 04
9	OWG 7



NOTE:  
Su: Undrained shear strength interpreted from piezo cone tests  
Information about boreholes drilled by PETO MACCALLUM LTD. were obtained from a report dated May 23, 2003  
See Sheet 5 and 6 for section locations.



## Appendix A

### Record of Borehole Sheets

## SYMBOLS AND TERMS USED ON TEST HOLE LOGS

### 1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

### 2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

### 3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT <sup>(1)</sup> "N" VALUE
Very Soft	Less than 10	Less than 2
Soft	10 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30







NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer


### 4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

### 5. LEGEND FOR TEST HOLE LOGS

SYMBOLS FOR	 Shelby Tube	 A - Casing
SAMPLE TYPE	 SPT	 Grab/Auger sample
	 No Recovery	 Core

- MC – Moisture Content (% by Weight) as determined by sample

 Water Level

$C_{vane}$	Shear Strength Determination by Field Insitu Vane
$C_{pen}$	Shear Strength Determination by Pocket Penetrometer
$C_{lab}$	Shear Strength Determination using a Laboratory Vane Apparatus
$C_U$	Undrained Shear Strength determined by Unconfined Compression Test


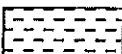



- (1) SPT Standard Penetration Test – refers to the number of blows from a 63.5kg hammer falling through 0.76m to advance a 60 degree truncated cone 0.3m.



# UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ( $W_L < 30\%$ ).
		CI	Inorganic clays of medium plasticity, silty clays. ( $30\% < W_L < 50\%$ ).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
	HIGHLY ORGANIC SOILS		Pt
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

## EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION		SYMBOLS	
Fresh (FR)	No visible signs of weathering.		
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.		CLAYSTONE
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.		COAL
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
		Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
		Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

TERMS	
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

# RECORD OF BOREHOLE No BH-1S

1 OF 1

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 613.57 E 318 462.23 ORIGINATED BY GA  
 HWY 69 BOREHOLE TYPE Test Pit COMPILED BY WM  
 DATUM Geodetic DATE 12.05.04 - 12.05.04 CHECKED BY PJB

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
223.0													
222.8	TOPSOIL												
0.1	SAND, trace silt, trace rootlets		1	AS									
222.6	Brown												
0.4	Damp												
	REFUSAL AT 0.41 m. BEDROCK AT 0.41 m.												

ONTMT4 6415.GPJ 21/07/04

**METRIC**

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80			100
221.7 220.6 0.1	TOPSOIL (0.1 m) REFUSAL AT 0.1 m. BEDROCK AT 0.1 m.													
						221								

+ 3, × 3: Numbers refer to Sensitivity

**METRIC**

ELEV DEPTH	SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  Y  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20					
234.3													
							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
							20 40 60 80 100 20 40 60 80 100						
							WATER CONTENT (%) 20 40 60						
													GR SA SI CL

[illegible]

+ 3, x 3: Numbers refer to Sensitivity

ONTMT4 6415.GPJ 21/07/04

## METRIC

SOIL PROFILE				SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
225.5 225.4 0.1	TOPSOIL (0.05 m) REFUSAL AT 0.05 m. BEDROCK AT 0.05 m.													
							225							

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH-5S

1 OF 1

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 617.67 E 318 459.32 ORIGINATED BY GA  
 HWY 69 BOREHOLE TYPE Test Pit COMPILED BY WM  
 DATUM Geodetic DATE 12.05.04 - 12.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)
225.2 0.0							20	40	60	80	100	20	40	60			
	HOLE ON BEDROCK. BEDROCK EXPOSED.						225										

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+<sup>3</sup>.X<sup>3</sup>: Numbers refer to  
Sensitivity





20  
15 Φ 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-6S

1 OF 1

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 632.76 E 318 432.38 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 26.05.04 - 26.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
219.0								20	40	60	80	100					
0.0	PEAT Black (PT)																
218.3																	
0.7	SILT, trace sand, trace clay, occasional thin sand seams Compact Brown Wet (ML-NP)						218										
			1	SS	10		217										0 3 90 7
216.6																	
2.4	SAND, some silt, trace gravel Compact Brown Wet (SP)		2	SS	13		216										
216.0					FI												
3.0	CASING REFUSAL AT 3.02 m. CORING STARTED AT 3.02 m. GRANITE, fresh, medium grey with dark sub-horizontal bands, very strong to extremely strong sub-vertical joints at 3.84, 4.52, 4.67, 5.00, 5.89 and 5.99 m		1	RUN	1 0 1		215										RUN 1# TCR=100%, SCR=100%, RQD=90%, UCS=227.5MPa
					0 1 0 1 1												RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=259.6MPa
			2	RUN			214										
					0 1 0 1 1												
					0 2		213										RUN 3# TCR=100%, SCR=100%, RQD=100%, UCS=167.6MPa
212.7																	
6.3	END OF BOREHOLE AT 6.25 m.																

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No BH-7S

1 OF 1

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 636.85 E 318 429.49 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 27.05.04 - 27.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
							WATER CONTENT (%)							
							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W <sub>P</sub> W W <sub>L</sub>							
219.1							219							
0.0	PEAT Black (PT)													
218.4														
0.6	Silty SAND Compact Brown Wet (SM)						218							
			1	SS	15									
216.9							217							
2.1	Sandy SILT, trace clay, occasional sand seams Compact Brown Wet (ML-NP)		2	SS	10									0 27 66 7
							216							
215.2														
3.8	END OF BOREHOLE AT 3.81 m. CASING REFUSAL AT 3.81 m. PROBABLE BEDROCK OR BOULDER. HOLE CAVED TO 0.99 m. WATER LEVEL AT SURFACE ON COMPLETION OF DRILLING.													

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-8S

1 OF 1

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 638.49 E 318 436.57 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 27.05.04 - 27.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
219.1	PEAT Black (PT)						219							
218.3														
0.8	Sandy SILT Compact Brown Wet (ML-NONPLASTIC)						218							
			1	SS	11		217							
216.7														
2.4	Silty SAND Loose Brown Wet (SM)						216							
			2	SS	5									0 56 44 (SH+CL)
215.5														
3.6	END OF BOREHOLE AT 3.61 m. CASING REFUSAL AT 3.61 m. PROBABLE BEDROCK OR BOULDER. HOLE CAVED TO 0.91 m. WATER LEVEL ON COMPLETION OF DRILLING AT SURFACE.													

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-9S

1 OF 1

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 640.08 E 318 443.14 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 27.05.04 - 27.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	TN VALUES			20	40	60	80	100		
219.2	PEAT						219							
0.0	Black (PT)													
218.7	Silty CLAY, with silt seams						218							
0.5	Very Soft		1	SS	1		217							
	Brown (CL)													
	varved		2	SS	2		216							
215.7														
3.5	SILT, some clay, trace sand, trace gravel						215							
	Compact		3	SS	11									
	Brown													
	Wet (ML)						214							
213.7			4	SS	50									
5.6	END OF BOREHOLE AT 5.56 m. CASING REFUSAL AT 5.56 m. PROBABLE BEDROCK OR BOULDER. WATER LEVEL ON COMPLETION OF DRILLING AT SURFACE.				.075									

+ 3, x 3: Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

ONTMT4 6415.GPJ 21/07/04

# RECORD OF BOREHOLE No BH-10S

1 OF 1

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 644.11 E 318 440.28 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 27.05.04 - 27.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
219.2 0.0	PEAT Black (PT)													
218.1 1.1	Silty CLAY Firm to Soft Grey (CL)		1	SS	5									
	varved, occasional silt lenses		2	SS	2									
215.2 4.0	SILT, trace clay Compact Grey (ML-NONPLASTIC)		3	SS	14									
213.7 5.5	SAND and GRAVEL, some silt, occasional cobbles Very Dense Grey Wet		4	SS	68/ 250									
212.6 6.6	END OF SOIL SAMPLING AT 6.58 m. CORING STARTED AT 6.58 m. GRANITE, slightly weathered, grey, strong to extremely strong sub-vertical joints at 7.19, 7.24, 7.44, 7.56, 8.28, 8.53, 8.71, 9.07, 9.14 and 9.22 m horizontal joints at 7.37, 8.18 and 9.45 m		1	RUN	>5									
			2	RUN	3									
			3	RUN	2									
			4	RUN	1									
209.7 9.6	END OF BOREHOLE AT 9.55 m.													

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

ONTMT4 6415.GPJ 21/07/04

RUN 2#  
TCR=96%,  
SCR=93%,  
RQD=76%,  
UCS=110.6MPa

RUN 3#  
TCR=97%,  
SCR=93%,  
RQD=79%,  
UCS=249.9MPa

# RECORD OF BOREHOLE No BH-11S

1 OF 3

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 666.34 E 318 410.91 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 28.05.04 - 28.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				20 40 60 80 100 20 40 60						
219.2 0.0	PEAT, some sand, trace silt Black (PT)															
			1	SS	WH											
			2	SS	WH											
			3	SS	WH											
214.3 4.9	Silty CLAY, occasional thin silt seams Soft to Firm Grey (CL-CI)		4	SS	WH											
			5	SS	2											
			1	TW	PH											

Continued Next Page

+ 3, × 3 : Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

ONTMT4 6415.GPJ 28/07/04

### METRIC

[illegible]

+ 3, x 3: Numbers refer to Sensitivity

ONTMT4 6415.GPJ 28/07/04

**METRIC**[illegible]

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-12S

1 OF 3

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 673.16 E 318 416.36 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 29.05.04 - 29.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
219.3 0.0	PEAT Black (PT)													
			1	SS	0		219						54	
			2	SS	0		218						61	
			3	SS	0		217							
							216							
							215						25	
213.8 5.5	Silty CLAY, trace sand Soft to Stiff Grey (CL-CI)		4	SS	2		214							
							213	12						
			5	SS	1		212						6	0 4 37 59
							211	9						
	becoming more plastic (CI-CH)		6	SS	2		210							
								10						

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

ONTMT4 5415.GPJ 21/07/04



## METRIC

ELEV DEPTH	SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	w <sub>p</sub>	w			w <sub>L</sub>
								SHEAR STRENGTH kPa	WATER CONTENT (%)				
							○ UNCONFINED + FIELD VANE						
							● QUICK TRIAXIAL × LAB VANE						
							20 40 60 80 100		20 40 60				

DEPTH (m)	LOG DESCRIPTION	UNIT	SS	FI	TEST DATA
203.5	occasional silt seams	7	SS	2	
		8	SS	1	
		9	SS	2	
		10	SS	4	
200.2	CASING REFUSAL AT 15.82 m. CORING STARTED AT 15.83 m. GRANITE, fresh, slightly weathered at joints, grey and occasional red and green subvertical banding, strong to very strong sub-vertical joints at 16.13, 16.26, 16.79, 16.84, 16.97, 17.02, 17.25, 17.30, 17.40, 17.43, 17.67, 17.78, 17.84, 18.31 and 18.38 m vertical joints at 16.41, 16.56, 18.54, 18.57, 18.95 and 18.99 m  rubble zone between 18.54 and 18.8 m	1	RUN	6	
		2	RUN	5	
		3	RUN	1	
		4	RUN	3	
		5	RUN	2	
		6	RUN	2	
19.1	END OF BOREHOLE AT 19.05 m. BOREHOLE BACKFILLED WITH BENTONITE SLURRY TO SURFACE.			12	
				5	

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Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No BH-12S

3 OF 3

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 673.16 E 318 416.36 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 29.05.04 - 29.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  Y kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
								20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	20 40 60 WP W WL					TCR=100%, SCR=55%, RQD=30%, UCS=175.28MPa

ONTMT4 6415.GPJ 21/07/04

+<sup>3</sup> ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-13S

1 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 700.88 E 318 385.65 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 28.05.04 - 29.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT      NATURAL MOISTURE LIMIT      LIQUID LIMIT		UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)			
219.1							20 40 60 80 100	20 40 60					
219.0	PEAT, black (PT)						20 40 60 80 100	20 40 60					
0.2	SILT, trace to some sand, trace to some clay Loose Brown Wet (ML)		1	SS	WH								0 16 87 17
			2	SS	5								
216.7													
2.4	SAND, medium grained, trace silt to silty Loose to Very Loose Brown Wet (SP/SM)		3	SS	9								
			4	SS	4								
	becoming grey												
			5	SS	2								
212.1													
7.0	Silty CLAY, occasional sand layers Firm to Stiff Grey (CL)		6	SS	WH								
							3.8						
			7	SS	1								

Continued Next Page

+<sup>3</sup> × 3<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-13S

2 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 700.88 E 318 385.65 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 28.05.04 - 29.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60					
209															
208															
207			8	SS	1										0 0 60 40
206															
205															
204			9	SS	6										
203															
202															
201.5															
17.7	Silty CLAY Firm to Stiff Grey (CH)		10	SS	4										
201															
200															

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Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity 20  
15 Φ 5  
10 (%) STRAIN AT FAILURE

## METRIC

DATUM	Geodetic	DATE	28.05.04 - 29.05.04	CHECKED BY	PJB
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(%) STRAIN AT FAILURE

## METRIC

[illegible]

(%) STRAIN AT FAILURE

ONTMT4 6415.GPJ 21/07/04

# RECORD OF BOREHOLE No BH-13S

5 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 700.88 E 318 385.65 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 28.05.04 - 29.05.04 CHECKED BY PJB

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
						20	40	60	80	100	20	40	60			
178.3			3	RUN	>5 >5 >5	179										GR SA SI CL RUN 3# TCR=80%, SCR=60%, RQD=52%, UCS=146.3MPa
40.9	END OF BOREHOLE AT 40.87 m. Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 3.05 m slotted screen.  WATER LEVEL READINGS: DATE DEPTH (m) 01/06/04 -1.12 (above ground surface)															

ONTMT4 6415.GPJ 21/07/04

+ 3 . x 3 : Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-14S

1 OF 5

METRIC

G.W.P. 312-89-00 LOCATION N 5 135 707.44 E 318 394.36 ORIGINATED BY WM  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 29.05.04 - 30.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
219.0	0.0 SAND, fine to medium grained, some silt to silty Brown Wet (FILL)						219					
217.5	1.5 SAND and SILT, fine to medium grained Loose		1	SS	7		218					
217.1	2.0 Brown Wet (SM/ML)						217					
216.4	2.6 SILT, trace sand, trace clay, occasional iron oxide staining Loose Brown (ML-NONPLASTIC)		2	SS	7		216					
	SAND, fine grained, some silt Loose Brown Wet (SP) becoming grey, trace silt		3	SS	4		215					
213.5	5.5 Silty CLAY Firm Grey (CL)		4	SS	1		214					
							213					
			5	SS	WH		212					0 0 68 32
							211	3.1				
			6	SS	WH		210	2.9				

ONTMT4 6415.GPJ 21/07/04

Continued Next Page

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15 10 5  
(%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No BH-14S

2 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 707.44 E 318 394.36 ORIGINATED BY WM  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 29.05.04 - 30.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%)					
209														
208														
207			7	SS	WH									0 0 64 36
206														
205														
204			8	SS	2									
203														
202														
201			1	TW	PH									
200.7														
18.3	Silty CLAY, occasional silt layers Firm to Stiff Grey (CI-CH)		9	SS	4									
200														

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15 10 5  
(%) STRAIN AT FAILURE

ONTM74 6415.GPJ 21/07/04

## METRIC

UNIM14 0413.GFJ 210/704

[illegible]

Continued Next Page

+ 3, × 3; Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

## METRIC

[illegible]

(%) STRAIN AT FAILURE

ONTMT4 6415.GPJ 21/07/04

# RECORD OF BOREHOLE No BH-14S

5 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 707.44 E 318 394.36 ORIGINATED BY WM  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 29.05.04 - 30.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W <sub>P</sub> W W <sub>L</sub>	WATER CONTENT (%)	20 40 60		
	BOREHOLE GROUTED WITH BENTONITE SLURRY TO SURFACE.													

ONTMT4 8415.GPJ 21/07/04

# RECORD OF BOREHOLE No BH-15S

1 OF 6

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 727.91 E 318 367.90 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE NW Casing/Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 26.05.04 - 27.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
223.4 223.8 0.1	TOPSOIL (100 mm) SAND, silty to trace silt, medium to coarse grained Compact to Loose Brown Moist to Wet (SP/SM)						223							
			1	SS	12		222							
			2	SS	9		221							
							220							
219.5 4.0	SILT, some clay to clayey, some sand to sandy, some clayey silt layers Loose Brown Wet (CL-ML/ML)		3	SS	4		219							
			4	SS	2		218							
							217							
			5	SS	9		216							
214.9 8.5	SAND, medium grained, trace silt Compact Brown Wet (SP)		6	SS	16		215							
							214							

Continued Next Page

+ 3, x 3 : Numbers refer to Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

ONTMT4 6415.GPJ 21/07/04

**METRIC**[illegible]

+ 3, X 3: Numbers refer to Sensitivity

CONTMT4 6415.GPJ 21/07/04

# RECORD OF BOREHOLE No BH-155

3 OF 6

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 727.91 E 318 367.90 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE NW Casing/Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 26.05.04 - 27.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							WATER CONTENT (%)
							20	40	60	80	100	20	40	60	
							</								

Continued Next Page

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

## METRIC

ORIGINATED BY JL

COMPILED BY WM

CHECKED BY          PJB

(%) STRAIN AT FAILURE



## METRIC

ORIGINATED BY JL

COMPILED BY WM

CHECKED BY         PJB        

ONTM T4 6415.GPJ 21/07/04

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-15S

6 OF 6

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 727.91 E 318 367.90 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE NW Casing/Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 26.05.04 - 27.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
172.2							173							
51.2	SAND & GRAVEL		20	SS	50 FI		172							
171.9														
51.5	CASING REFUSAL AT 51.51 m. CORING STARTED AT 51.51 m. GRANITE, fresh, dark grey with green and red subvertical banding, strong to extremely strong sub-vertical joints at 51.79, 51.84, 52.07, 52.40, 52.44, 52.48, 52.50, 53.44 and 53.49 m sub-horizontal joints at 52.43, 52.77, 53.10, 53.52 and 53.54 m  rubble zones from 53.49 to 53.52 m, 53.77 to 53.82 m, 53.97 to 54.03 m  mechanical breaks at 54.18 and 54.25 m		1	RUN	1		171							RUN 1# TCR=100%, SCR=78%, RQD=56%, UCS=125.2MPa RUN 2# TCR=93%, SCR=93%, RQD=63%, UCS=267.1MPa RUN 3# TCR=100%, SCR=91%, RQD=91%, UCS=82.1MPa RUN 4# TCR=100%, SCR=89%, RQD=77%, UCS=112MPa
			2	RUN	2									
			3	RUN	1									
			4	RUN	5									
168.8							170							
54.6	END OF BOREHOLE AT 54.61 m. Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 3.05 m slotted screen.  WATER LEVEL READINGS: DATE DEPTH 01/06/04 (m) 3.09						169							

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-16S

1 OF 6

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 735.20 E 318 378.72 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 20.05.04 - 21.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT  $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE		WATER CONTENT (%) W <sub>P</sub> W W <sub>L</sub>				
223.4							20 40 60 80 100							
220.8	TOPSOIL (125 mm)						20 40 60 80 100							
0.1	SAND, medium grained, trace to some silt Compact Brown Wet (SP)													
	some silt and clay layers		1	SS	14									
			2	SS	12									
219.5														
4.0	Sandy SILT, some clay, occasional sand seams Compact to Loose Brown Wet (ML)		3	SS	6									
217.9														
5.5	SILT, clayey to some clay, sandy, occasional sand seams, occasional oxide staining Stiff to Very Stiff Brown (CL-ML/ML)		4	SS	12									
	frequent sand layers		5	SS	26									0 26 65 9
214.9														
8.5	SAND, medium grained, trace silt Compact Brown Wet (SP)		6	SS	15									

Continued Next Page

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15 10 5  
(%) STRAIN AT FAILURE

ONTMT4 8415.GPJ 21/07/04

# RECORD OF BOREHOLE No BH-16S

2 OF 6

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 735.20 E 318 378.72 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 20.05.04 - 21.05.04 CHECKED BY PJB

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
211.9	Silty CLAY Stiff to Firm Grey (CL)		7	SS	1								
212													
211													
210													
209													
208	some silt seams		8	SS	WH								0 0 59 41
207													
206													
205													
204													

Continued Next Page

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15-5  
10

(%) STRAIN AT FAILURE

ONTMT4 6415.GPJ 21/07/04

**METRIC**

DN7MT4 6415.GPJ 21/07/04

+ 3, X 3: Numbers refer to Sensitivity



### METRIC

CONTMT4 6415.GPJ 21/07/04

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No BH-16S

6 OF 6

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 735.20 E 318 378.72 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 20.05.04 - 21.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	*N VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)						
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL      × LAB VANE					w <sub>p</sub> w      w <sub>L</sub>						
							20	40	60	80	100	20	40	60	GR	SA	SI	CL	
173.3																			
50.1	CASING REFUSAL AT 50.14 m. CORING STARTED AT 50.14 m. boulder from 50.14 to 50.47 m cobbles and boulders from 50.47 to 51.0 m		1	RUN	FI		173												RUN 2# TCR=74%, SCR=90%, RQD=41%, UCS=128.3MPa
172.4																			
51.0	GRANITE, fresh to slightly weathered, grey with occasional red and dark grey banding, very strong to extremely strong sub-horizontal joints at 51.10, 51.21, 51.28, 51.46, 51.59 and 51.73 m sub-vertical joints at 51.82 and 51.89 m sub-horizontal joints at 52.02, 52.35 and 53.26 m  sub-vertical joints at 52.68, 52.73, 52.88 and 52.91 m		2	RUN	4		172												RUN 3# TCR=100%, SCR=100%, RQD=75%, UCS=183.6MPa
																			</

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15-0.5  
10

(%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No BH-17S

1 OF 6

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 748.97 E 318 382.46 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE Hollow Stem Augers / Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 18.05.04 - 19.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100							WATER CONTENT (%) Wp — W — WL 20 40 60
224.1 0.0 223.9	TOPSOIL (225 mm)						224								
0.2	SAND, medium grained, trace silt Compact Brown Wet (SP)						223								
			1	SS	18		222								
			2	SS	13		221								
220.1							220								
4.0	Sandy SILT Compact Brown Wet (ML-NONPLASTIC)		3	SS	10		219								
218.6							218								
5.5	Clayey SILT, some sand layers Soft Brown (CL-ML)		4	SS	3		217								
217.1							216								
7.0	SAND, medium grained, trace silt Compact Brown Wet (SP)		5	SS	15		215								
216.6							214								
7.5	Sandy SILT, trace clay Compact Brown (ML-NONPLASTIC)						213								
215.6							212								
8.5	Silty CLAY, trace sand Stiff Brown (CL)		6	SS	10		211								
215.1							210								
9.0	SAND, medium grained, trace silt Compact Brown Wet (SP)						209								

ONTMT4 6415.GPJ 21/07/04

Continued Next Page

+ 3, × 3 : Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE



**METRIC**

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT 	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI
ELEV DEPTH	DESCRIPTION	STRAT PLOT NUMBER	TYPE						
						SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	WATER CONTENT (%) 20 40 60		

**Geotechnical Log Data:**

Depth (m)	Soil Description	Soil Type	Notes
199.6 - 204.0	Silty CLAY Stiff Grey (CI-CH)	13 SS	4
200.0	TW	PH	
200.0 - 204.0	SS	14	5
204.0 - 208.0	SS	15	3

**Graph Data (Approximate values from plot):**

Depth (m)	Water Content (w) (%)	Liquid Limit (LL) (%)
204.0	~28	~28
203.0	~28	~28
202.0	~28	~28
201.0	~28	~28
200.0	~28	~28
199.0	~28	~28
198.0	~28	~28
197.0	~28	~28
196.0	~28	~28
195.0	~28	~28

**Soil Properties:**

- $e_s = 0.949$
- $C_v = 12.4 \text{ m}^2/\text{s}$
- $G = 2.76$
- $C_u = 0.32$

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

CONTMT4 6415.GPJ 21/07/04

# RECORD OF BOREHOLE No BH-17S

4 OF 6

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 748.97 E 318 362.46 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE Hollow Stem Augers / Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 18.05.04 - 19.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
			16	SS	7		194							0 1 31 68
							193							
							192							
			2	TW	PH		191							
							190							
							189							
			17	SS	7		188							0 1 24 75
							187							
							186							
185.1							185							
39.0	SILT, some clay, trace sand Firm to Soft Grey (ML)		18	SS	7									

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+ 3, x 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

ONTMT4 6415.GPJ 21/07/04

# RECORD OF BOREHOLE No BH-17S

5 OF 6

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 748.97 E 318 362.46 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE Hollow Stem Augers / Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 18.05.04 - 19.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)		
								20	40	60			80	100	20
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT w <sub>p</sub> w w <sub>L</sub>					
							184								
							183								
			19	SS	1		182							0 1 88 11	
							181								
							180								
			20	SS	16		179								
							178								
							177								
175.9							176								
48.2	Sandy SILT, occasional sand layers, trace clay Compact Grey Wet (ML-NONPLASTIC)		21	SS	24		175							0 30 61 9	

ONTMT4 6415.GPJ 21/07/04

ONTMT4 6415.GPJ 21/07/04

Continued Next Page

+<sup>3</sup> × 3: Numbers refer to  
Sensitivity

20  
15 10 5  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-17S

6 OF 6

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 748.97 E 318 362.46 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE Hollow Stem Augers / Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 18.05.04 - 19.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
172.8			22	SS	50/		174										
51.3	END OF BOREHOLE AT 51.33 m. SPOON REFUSAL AT 51.33 m. PROBABLE BOULDER OR BEDROCK AT 51.33 m.				.0		173										

ONTMT4 6415.GPJ 21/07/04

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-18S

1 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 796.14 E 318 333.32 ORIGINATED BY GA/SL  
 HWY 69 BOREHOLE TYPE Hollow Stem Augers / NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 13.05.04 - 15.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
								20 40 60 80 100										
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										
224.4 0.0	Silty SAND mixed with TOPSOIL, trace rootlets, trace organics Compact Brown		1	SS	12													
222.9 1.5	SILT, trace sand to sandy, trace to some clay, occasional iron oxide staining Compact Brown (ML-NONPLASTIC)		2	SS	10													
			3	SS	10											0 23 67 10		
219.9 4.6	SILT, some sand, trace clay Compact Brown (ML-NONPLASTIC)		4	SS	10													
			5	SS	15											0 6 80 15		
	occasional sand seams																	
216.8 7.6	Clayey SILT, trace sand, occasional silt lenses, occasional sand lenses Stiff to Firm Grey (CL-ML)		6	SS	8													
			7	SS	6													

ONTM14 6415.GPJ 21/07/04

Continued Next Page

+ 3, × 3: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

ONTMT4 6415.GPJ 21/07/04





# RECORD OF BOREHOLE No BH-18S

3 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 796.14 E 318 333.32 ORIGINATED BY GAVSL  
 HWY 69 BOREHOLE TYPE Hollow Stem Augers / NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 13.05.04 - 15.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
			14	SS	WH		20 40 60 80 100	20 40 60						
							204							
								3.1						
							203							
							202							
			15	SS	WH		201							
							200							
							199							
198.5														
25.9	Silty CLAY Stiff to Firm Grey (CI-CH)		16	SS	6		198							
								3.3						
							197							
			1	TW	PH		196							
			17	SS	5		195							
								2.7						
														0 0 19 81

ONTMT4 6415.GPJ 21/07/04

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

**METRIC**

CONTMT4 6415.GPJ 21/07/04

Continued Next Page

+ 3, x 3: Numbers refer to Sensitivity

## METRIC



ONTMT4 6415.GPJ 21/07/04

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

## METRIC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT 	UNIT WEIGHT  γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
			NUMBER	TYPE						
							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	WATER CONTENT (%) 20 40 60		

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

ONTMT4 6415.GPJ 21/07/04

# RECORD OF BOREHOLE No BH-2N

1 OF 1

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 645.04 E 318 470.24 ORIGINATED BY GA  
 HWY 69 BOREHOLE TYPE Test Pit COMPILED BY WM  
 DATUM Geodetic DATE 12.05.04 - 12.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)						
						20	40	60	80	100	20	40	60				
223.4	TOPSOIL (0.09 m)																
228.8	REFUSAL AT 0.09 m.																
0.1	BEDROCK AT 0.09 m.																

ONTMT4 6415.GPJ 21/07/04

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-3N

1 OF 1

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 646.67 E 318 477.06 ORIGINATED BY GA  
 HWY 69 BOREHOLE TYPE Test Pit COMPILED BY WM  
 DATUM Geodetic DATE 12.05.04 - 12.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)							
223.6																		
223.6	TOPSOIL (0.076 m)																	
0.1	SILT, trace sand, trace clay, occasional rootlets																	
223.3																		
0.4	Brown Damp																	
	REFUSAL AT 0.36 m. BEDROCK AT 0.36 m.																	

ONTMT4 6415.GPJ 21/07/04

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity  
 20  
 15  
 10  
 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-4N

1 OF 1

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 648.26 E 318 484.12 ORIGINATED BY GA  
 HWY 69 BOREHOLE TYPE Test Pit COMPILED BY WM  
 DATUM Geodetic DATE 12.05.04 - 12.05.04 CHECKED BY PJB

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
223.6 223.0 0.1	TOPSOIL (0.076 m) REFUSAL AT 0.076 m. BEDROCK AT 0.076 m.						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	20 40 60					
						223							

ONTMT4 6415.GPJ 21/07/04

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity 20 15 10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-5N

1 OF 1

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 652.40 E 318 481.14 ORIGINATED BY GA  
 HWY 69 BOREHOLE TYPE Test Pit COMPILED BY WM  
 DATUM Geodetic DATE 12.05.04 - 12.05.04 CHECKED BY PJB

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
						20	40	60	80	100						
223.3																
223.8	TOPSOIL (0.09 m)															
0.1	REFUSAL AT 0.09 m. BEDROCK AT 0.09 m.					223										

ONTMT4 6415.GPJ 21/07/04

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No BH-6N

1 OF 2

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 673.63 E 318 450.12 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 03.06.04 - 03.06.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
219.0 0.0	PEAT Black Wet (PT)						219	20	40	60	80	100	20	40	60	GR SA SI CL
217.8								218								
1.2	Silty CLAY, trace sand Firm to Soft Grey (CL)		1	SS	WH		217									0 3 64 33
			2	SS	1		216									
							215									2 72 26 (SI+CL)
			3	SS	3		214									
							213									
213.0			4	SS	3		212									
6.0	SAND, some silt to silty, trace gravel, occasional clay lumps Loose Grey Wet (SP/SM)		5	SS	9		211									
							210									
			6	SS	4											

Continued Next Page

+ 3, X 3: Numbers refer to  
Sensitivity 20  
15 10 5 (%) STRAIN AT FAILURE

ONTMT4 6415 GPJ 21/07/04

## METRIC

[illegible]

+ 3, x 3: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No BH-7N

1 OF 2

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 680.97 E 318 461.00 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 03.06.04 - 03.06.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE						
219.2 0.0	PEAT Black Wet (PT)						20 40 60 80 100	20 40 60 80 100							
217.4 1.8	Silty CLAY, trace sand Stiff to Soft Grey (CL)		1	SS	1										
			2	SS	1										
			3	SS	2										
			4	SS	2										
			1	TW	PH										
	Becoming Cl		5	SS	4										
210.7 8.5	SILT, trace clay, trace sand Compact Grey Wet (ML-NONPLASTIC)		6	SS	26										
209.3															

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

ONTMT4 6415.GPJ 21/07/04

# RECORD OF BOREHOLE No BH-7N

2 OF 2

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 680.97 E 318 461.00 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 03.06.04 - 03.06.04 CHECKED BY PJB

ONTMT4 6415.GPJ 21/07/04

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa												
							○ UNCONFINED + FIELD VANE					● QUICK TRIAXIAL × LAB VANE							
							20 40 60 80 100					20 40 60							
10.0	CASING REFUSAL AT 9.99 m. CORING STARTED AT 9.99 m. GRANITE, fresh, slightly to medium weathered, grey, strong to very strong sub-vertical joints at 10.06, 10.21 m horizontal joint at 10.52 m horizontal joint at 10.74 m  horizontal joint at 11.23, 11.99 m  sub-vertical joints at 11.6, 11.71, 11.86 m  sub-vertical joints at 12.36, 12.60, 12.93 m		1	RUN	2 3		209									RUN 1# TCR=100%, SCR=100%, RQD=69%, UCS=64.7MPa			
			2	RUN	1 0		208									RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=189.2MPa			
			3	RUN	2 2 3 0		207									RUN 3# TCR=100%, SCR=95%, RQD=90%, UCS=187.3MPa			
			4	RUN	1 2 3											RUN 4# TCR=100%, SCR=100%, RQD=100%, UCS=229.2MPa			
206.1																			
13.1	END OF BOREHOLE AT 13.13 m. BOREHOLE GROUTED WITH BENTONITE SLURRY TO SURFACE.																		

+ 3, × 3: Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-8N

1 OF 3

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 706.69 E 318 427.42 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 08.06.04 - 08.06.04 CHECKED BY PJB

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20					
219.2 0.0	PEAT Black Wet (PT)												
			1	SS	WH								
			2	SS	WH								
			3	SS	WH								
213.4 5.8	Silty CLAY Soft to Stiff Grey (CL)		4	SS	1								
			5	SS	3								
			6	SS	5								

Continued Next Page

+<sup>3</sup> × 3<sup>3</sup>: Numbers refer to Sensitivity 20 15 10 6 (%) STRAIN AT FAILURE

ONTMT4 6415.GPJ 21/07/04



# RECORD OF BOREHOLE No BH-8N

3 OF 3

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 706.69 E 318 427.42 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 08.06.04 - 08.06.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
197.1	occasional silt layers becoming very stiff		10	SS	18									
22.1	CASING REFUSAL AT 22.13 m.													
196.8	CORING STARTED AT 22.13 m.													
22.4	BOULDER (0.30 m)		1	RUN	FI									
196.6	SAND (0.33 m)													
22.8	GRANITE, fresh to slightly weathered, grey with occasional pink speckles, medium to very strong horizontal joints at 22.8, 23.29 m sub-vertical joints at 22.91, 22.99, 23.16 m Diorite layer from 22.91 to 22.98 m horizontal joint at 23.42 m sub-vertical joints at 23.62, 23.67, 23.77, 23.98, 24.18 m Diorite layer from 24.11 to 24.23 m		2	RUN	3									
			3	RUN	3									
			4	RUN	2									
	sub-vertical joints at 24.84, 25.06, 25.12, 25.43, 25.63, 25.81 m													
193.4														
25.8	END OF BOREHOLE AT 25.83 m. Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 1.52 m slotted screen.													
	WATER LEVEL READINGS: DATE DEPTH (m) 08/06/04 0 (at surface)													



ONTMT4 6415.GPJ 21/07/04

# RECORD OF BOREHOLE No BH-9N

1 OF 3

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 712.04 E 318 435.44 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 03.06.04 - 03.06.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE						
219.2 0.0	PEAT Black Wet (PT)						219								
			1	SS	WH		218								
			2	SS	WH		217								
							216								
			3	SS	WH		215						350		
214.1 5.0	Silty CLAY, trace sand Firm to Stiff Grey (CL)		4	SS	1		214								
							213	1.4							
			5	SS	1		212								
							211	7.2							
	occasional sand seams		6	SS	5		210							0 3 55 42	

ONTMT4 6415.GPJ 21/07/04

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

ONTMT4 6415.GPJ 21/07/04



**METRIC**

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20				40	60	80	100	Wp	W	Wl
							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE											
							20	40	60	80	100							

203.9

15.2

Silty CLAY, trace sand  
Firm to Stiff  
Grey  
(CH)

Interval (ft)	Soil Type	Notes
7	SS	5
1	TW	PH
8	SS	3
9	SS	8

209

208

207

206

205

204

203

202

201

200

29

No Recovery  
from TW#2

0 2 2

ONTMT4 6415.GPJ 21/07/04

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-9N

3 OF 3

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 712.04 E 318 435.44 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 03.06.04 - 03.06.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)						
								○ UNCONFINED    + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE											
							20	40	60	80	100	20	40	60		GR	SA	SI	CL
198.4																			
20.7	SILT, trace clay		10	SS	69/														
198.0	Dense				250														
21.1	Grey (ML-NONPLASTIC)				FI														
	END OF SOIL SAMPLING AT 21.13 m.				2														
	CORING STARTED AT 21.14 m.				1														
	GRANITE, slightly weathered to fresh, light grey with occasional dark grey banding, strong to very strong sub-vertical joints at 21.51, 21.64, 21.67, 22.17, 22.33, 22.45 m horizontal joint at 21.98 m mechanical break at 22.58 m		1	RUN	2														
					2														
					2														
					2														
					2														
	sub-vertical joints at 22.99, 23.06, 23.16 m		2	RUN	2														
	sub-vertical joints at 23.16, 23.29, 23.34, 23.52, 23.55, 23.70, 23.77 m				2														
					2														
					2														
					2														
					3														
					3														
					1														
194.8																			
24.4	END OF BOREHOLE AT 24.36 m. BOREHOLE GROUTED WITH BENTONITE SLURRY TO SURFACE.																		

ONTMT4 6415.GPJ 21/07/04

RUN 1#  
 TCR=89%,  
 SCR=86%,  
 RQD=79%,  
 UCS=93.9MPa

RUN 2#  
 TCR=100%,  
 SCR=100%,  
 RQD=100%,  
 UCS=171.8MPa

RUN 3#  
 TCR=100%,  
 SCR=98%,  
 RQD=86%,  
 UCS=209.2MPa

ONTMT4 6415.GPJ 21/07/04

# RECORD OF BOREHOLE No BH-10N

1 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 238.40 E 318 403.54 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 31.05.04 - 01.06.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
220.3 0.0	SAND, medium grained Brown (FILL)						20 40 60 80 100	20 40 60 80 100						
219.3 0.9	SILT, sandy, clayey Loose Brown Wet (ML-NONPLASTIC)		1	SS	7									
			2	SS	2									0 22 54 24
216.3 4.0	SAND, medium grained, some silt Compact to Loose Brown Wet (SP)  Becoming grey		3	SS	14									
			4	SS	4									0 87 13 (SI+CL)
213.2 7.0	Sandy SILT, fine grained Very Loose Grey Wet (ML-NONPLASTIC)		5	SS	1									
211.7 8.5	Silty CLAY, occasional sand seams Very Soft Grey (CL)		6	SS	1									

ONTMT4 6415.GPJ 21/07/04

Continued Next Page

+ 3, × 3: Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

ONTMT4 6415.GPJ 21/07/04

## METRIC

CONTMT4 6415.GPJ 21/07/04

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-10N

3 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 238.40 E 318 403.54 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 31.05.04 - 01.06.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	varved, reddish brown seams		10	SS	4		200							
							199							
							198							
							197							
			11	SS	5		196							0 1 17 83
							195							
							194							
							193							
			12	SS	7		192							
							191							

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity 20 15 10 (%) STRAIN AT FAILURE

ONTMT4 8415.GPJ 21/07/04

# RECORD OF BOREHOLE No BH-10N

4 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 238.40 E 318 403.54 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 31.05.04 - 01.06.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
			13	SS	9		190							
							189							
							188							
			14	SS	4		187							
							186							
							185							
			15	SS	12		184							
							183							
182.3							182							
38.0	SILT, some to trace clay Compact Grey Wet (ML/ML-NONPLASTIC)						181							
			16	SS	22									

ONTMT4 6415.GPJ 21/07/04

Continued Next Page

+ 3 . × 3 : Numbers refer to  
Sensitivity

20  
15 Φ 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-10N

5 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 238.40 E 318 403.54 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 31.05.04 - 01.06.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
179.1							180							RUN 1# TCR=93%, SCR=50%, RQD=35%, UCS=123.6MPa  RUN 2# TCR=93%, SCR=60%, RQD=50%, UCS=192.0MPa
41.2	CASING REFUSAL AT 41.17 m. CORING STARTED AT 41.17 m. GRANITE, fresh, grey with dark grey, green and red sub-horizontal banding, high quartz content, very strong  rubble zone from 42.70 to 43.54 m  sub-vertical joints at 43.54, 43.61, 43.66, 43.74, 43.94, 44.17 m		1	RUN	5 4 4 3 1		179							
			2	RUN	1 1 4 >25 >25		178							
							177							
176.0														
44.2	END OF BOREHOLE AT 44.22 m. Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 3.05 m slotted screen.  WATER LEVEL READINGS: DATE DEPTH 08/06/04 2.1													

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity 20 15 10 (%) STRAIN AT FAILURE

ONTMT4 6415.GPJ 21/07/04





# RECORD OF BOREHOLE No BH-11N

2 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 750.44 E 318 407.54 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 30.05.04 - 31.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
	occasional silt layers		6	SS	3		210							
							209							
							208							
							207							
			7	SS	1		206							
							205							
							204							
			1	TW	PH		203							
202.7							202							
18.3	Silty CLAY, occasional silt layers Stiff Grey (CH)		8	SS	2		201							

ONTMT4 6415.GPJ 21/07/04

Continued Next Page

+ 3, X 3: Numbers refer to  
Sensitivity

20  
15 10 5  
(%) STRAIN AT FAILURE

**METRIC**

CHECKED BY           PJB          

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-11N

4 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 750.44 E 318 407.54 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 30.05.04 - 31.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)						
								○ UNCONFINED   + FIELD VANE							w <sub>p</sub> w   w <sub>L</sub>						
								● QUICK TRIAXIAL   × LAB VANE													
							20	40	60	80	100	20	40	60		GR	SA	SI	CL		
			12	SS	6									○							
							190														
							189														
							188								┌──○──┐			0	0	23	77
			13	SS	15																
							187														
							186														
185.3							185							○							
35.7	SILT, some to trace clay, trace sand Compact Grey (ML-NONPLASTIC)		14	SS	16																
							184														
							183														
							182														
														○							
			15	SS	34																
							181														

Continued Next Page

+ 3 × 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

ONTMT4 6415.GPJ 21/07/04

# RECORD OF BOREHOLE No BH-11N

5 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 750.44 E 318 407.54 ORIGINATED BY JL  
HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
DATUM Geodetic DATE 30.05.04 - 31.05.04 CHECKED BY PJB

DATUM

Geodetic

CORRECTION

SOIL PROFILE			SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	20 40 60 80 100			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100			
179.0					FI										
42.0	CASING REFUSAL AT 41.96 m. CORING STARTED AT 41.97 m. GRANITE, fresh, grey with dark grey and red sub-horizontal banding, strong to very strong horizontal joints at 42.03, 42.34 m sub-vertical joint at 42.42 m rubble zone from 43.03 to 43.10 m vertical joint 43.10 to 43.26 m  rubble zone from 43.82 to 43.92 m  vertical joint from 44.58 to 44.88 m		1	RUN	3 2										RUN 1# TCR=100%, SCR=75%, RQD=54%, UCS=149.2MPa RUN 2# TCR=100%, SCR=43%, RQD=27%, UCS=105.0MPa
			2	RUN	4 4 5 5 >25										
			3	RUN	4 6 3 >25										RUN 3# TCR=100%, SCR=27%, RQD=27%, UCS=187.8MPa
175.6															
45.3	END OF BOREHOLE AT 45.34 m. BOREHOLE GROUTED WITH BENTONITE SLURRY TO SURFACE.														

ONTMT4 6415.GPJ 21/07/04

ONTMT4 6415.GPJ 21/07/04

+ 3, X 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-12N

1 OF 6

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 769.65 E 318 386.48 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE Hollow Stem Augers / Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 17.05.04 - 19.05.04 CHECKED BY PJB

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	WATER CONTENT (%)		
224.2 224.0 0.1	TOPSOIL (100 mm) SAND, medium to coarse grained, trace to some silt Compact to Loose Brown Wet (SP)		1	AS		224						
			1	SS	18	223						
			2	SS	7	221						0 91 9 (SI+CL)
219.6 4.6	SILT, sandy to trace sand, trace clay, occasional iron oxide staining Loose to Compact Brown Wet (ML-NONPLASTIC)		3	SS	9	219						
			4	SS	8	218						
			5	SS	18	216						
215.1 9.1	SAND, medium grained, trace silt Compact Brown Wet (SP)		6	SS	23	215						0 93 7 (SI+CL)

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

ONTMT4 6415.GPJ 21/07/04

**METRIC**[illegible]

+ 3, × 3: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No BH-12N

3 OF 6

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 769.65 E 318 386.48 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE Hollow Stem Augers / Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 17.05.04 - 19.05.04 CHECKED BY PJB

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
	Becoming Stiff		10	SS	13								
200.4													
23.8	Silly CLAY, trace sand Stiff to Very Stiff Grey (CI-CH)		11	SS	14								0 1 34 65
			12	SS	10								
194.3													

Continued Next Page

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

ONTM/T4 6415.GPJ 21/07/04

# RECORD OF BOREHOLE No BH-12N

4 OF 6

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 769.65 E 318 386.48 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE Hollow Stem Augers / Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 17.05.04 - 19.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
29.9	Silty CLAY Very Stiff to Stiff Grey (CI-CH)		13	SS	16		194							
			14	SS	15		191							0 0 14 86
			15	SS	14		188							
186.4	Clayey SILT Very Stiff Grey (CL-ML)		16	SS	18		185							

ONTMT4 6415.GPJ 21/07/04

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15 10 5  
(%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No BH-12N

5 OF 6

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 769.65 E 318 386.48 ORIGINATED BY JL  
 HWY 69 BOREHOLE TYPE Hollow Stem Augers / Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 17.05.04 - 19.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40					
182.1														
42.1	SILT, trace clay, occasional sand seams Compact to Dense Grey Wet (ML)		17	SS	30									0 0 90 10
			18	SS	38									
176.2					FI									
48.0	CASING REFUSAL AT 48.03 m. CORING STARTED AT 48.03 m. GRANITE, fresh, slightly weathered, light grey, medium strong to very strong rubble zone from 48.21 to 48.34 m sub-horizontal joint at 48.69 m sub-vertical joints at 48.72, 49.15 m sub-horizontal joints at 49.28, 49.40, 49.68, 49.76 m		1	RUN	>5									RUN 1# TCR=100%, SCR=67%, RQD=38%, UCS=214.9MPa
			2	RUN	1									RUN 2# TCR=100%, SCR=100%, RQD=62%, UCS=131.5MPa
			3	RUN	2									RUN 3# TCR=100%,
					2									

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+ 3, x 3 : Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

ONTMT4 8415.GPJ 21/07/04



**METRIC**

ORIGINATED BY SL

COMPILED BY WM

CHECKED BY PJB

ONTMT4 6415.GPJ 21/07/04

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-13N

2 OF 6

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 776.52 E 318 397.21 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 19.05.04 - 20.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
212.4	Silty SAND Compact Brown-Grey Wet (SM)		7	SS	20		214							
11.6							213							
							212							
							211							
							210							
209.4	Silty CLAY Soft to Firm Grey (CL)		8	SS	2		209							
14.6							208							
							207							
			1	TW	PH		206							
			9	SS	3		205							
							204							

ONTMT4 6415.GPJ 21/07/04

Continued Next Page

+ 3, x 3; Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE





**METRIC**

CHECKED BY PJB

Continued Next Page

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-13N

6 OF 6

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 776.52 E 318 397.21 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 19.05.04 - 20.05.04 CHECKED BY PJB

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			*N* VALUES	SHEAR STRENGTH kPa								
						20	40	60	80	100						
173.8	50.19 m				2											
50.2	END OF BOREHOLE AT 50.19 m. BOREHOLE GROUTED WITH BENTONITE SLURRY TO SURFACE.															

ONTMT4 6415.GPJ 21/07/04



# RECORD OF BOREHOLE No BH-14N

1 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 794.93 E 318 377.88 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing/Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 16.05.04 - 17.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								20 40 60 80 100										
224.3																		
224.0	TOPSOIL (150 mm)																	
0.2	SAND, some silt to silty Compact Brown Moist to Wet (SM)						224											
			1	SS	13		223											
							222											
			2	SS	10		221											
							220											
219.8							219											
4.6	Sandy SILT, trace clay Loose to Compact Brown Wet (ML-NONPLASTIC)		3	SS	9		218											
							217											
217.9			4	SS	17		216											
6.4	SAND, medium grained, trace silt Compact Brown Moist (SP)						215											
216.3			5	SS	17													
8.0	SILT, trace sand Compact Brown Moist (ML-NONPLASTIC)																	
215.2																		
9.1	SAND, fine grained, trace silt Compact to Dense Brown Wet (SP)		6	SS	12													

ONTMT4 64115.GPJ 21/07/04

Continued Next Page

+ 3, × 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

ONTMT 6415.GPJ 21/07/04

# RECORD OF BOREHOLE No BH-14N

2 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 794.93 E 318 377.88 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing/Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 16.05.04 - 17.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>		
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%) 20 40 60				
212.6			7	SS	30		214						
11.7	Silty SAND, fine grained Grey Loose Wet (SM)		8	SS	8		213						
211.1							212						
13.3	Silty CLAY, trace sand Firm Grey (CL)		9	SS	1		211						
							210						
			10	SS	1		209	2.7					
							208						
			11	SS	1		207						0 0 62 38
							206						
			12	SS	2		205						

ONTMT4 6415.GPJ 21/07/04

Continued Next Page

+<sup>3</sup> ×<sup>3</sup>: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-14N

3 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 794.93 E 318 377.88 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing/Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 16.05.04 - 17.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
			1	TW	PH		204						18.9	C <sub>c</sub> =0.23 e <sub>s</sub> =0.908 C <sub>u</sub> =0.12 C <sub>r</sub> =31m <sup>2</sup> /y G=2.78
			13	SS	2		203							
							202							
							201							
			14	SS	1		200							
							199							
							198							
							197							
			2	TW	PH									
196.1							196							0 1 21 78
28.2	Silly CLAY (varved) Stiff Grey (CI-CH)		15	SS	6									
							195							

ONTMT4 6415.GPJ 21/07/04

Continued Next Page

+<sup>3</sup> ×<sup>3</sup>: Numbers refer to  
Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-14N

4 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 794.93 E 318 377.88 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing/Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 16.05.04 - 17.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80					
			16	SS	14											
			17	SS	13											0 0 27 73
			3	TW	PH											
			18	SS	9											
184.7																
39.6	Clayey SILT Firm		19	SS	6											0 0 80 20

Continued Next Page

+<sup>3</sup> × 3: Numbers refer to  
Sensitivity

20  
15-0-5  
10 (%) STRAIN AT FAILURE

ONTMT4 8415.GPJ 21/07/04

# RECORD OF BOREHOLE No BH-14N

5 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 794.93 E 318 377.88 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing/Wash Boring COMPILED BY WM  
 DATUM Geodetic DATE 16.05.04 - 17.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
181.7	Grey (CL-ML)						184							
42.7	SILT, trace clay Dense Grey (ML)		20	SS	35		183							
							182							
178.6							181							
45.7	Sandy SILT		21	SS	50/		180							
178.3	Very Dense						179							
46.0	Grey END OF BOREHOLE AT 46.02 m. CASING REFUSAL AT 46.02 m. BOREHOLE OPEN TO 45.11 m. PROBABLE BEDROCK OR BOULDER.													

+ 3, X 3: Numbers refer to  
Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

ONTMT4 8415.GPJ 21/07/04

# RECORD OF BOREHOLE No BH-15N

1 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 831.26 E 318 356.01 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 15.05.04 - 16.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
225.6																	
0.0	TOPSOIL (250 mm)																
225.3																	
0.3	Silty SAND Compact Brown Moist (SM)						225										
			1	SS	15		224										
							223										
222.5																	
3.1	Sandy SILT Compact to Loose Brown Wet (ML-NONPLASTIC)		2	SS	12		222										
							221										
			3	SS	23		220										
							219										
			4	SS	5												
218.2							218										
7.3	SAND, coarse grained, trace silt Dense Brown Wet (SP)		5	SS	38		217										
216.7							216										
8.8	Clayey SILT, trace gravel Hard Brown (CL-ML)		6	SS	34												

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+ 3, x 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

ONTMT4 6415.GPJ 21/07/04

## METRIC

ORIGINATED BY SL

COMPILED BY WM

CHECKED BY          PJB

ONTMT4 6415.GPJ 21/07/04

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-15N

3 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 831.26 E 318 356.01 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 15.05.04 - 16.05.04 CHECKED BY PJB

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
			1	TW	PH		○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
						205							
			13	SS	4	204							0 0 62 38
						203							
						202							
			14	SS	7	201							
						200							
						199							
198.7													
26.8	Silty CLAY, occasional silt lenses (varved) Soft to Stiff Grey (CI-CH)												
			15	SS	3	198							0 1 36 64
						197							
						196							

ONTMT4 6415.GPJ 21/07/04

Continued Next Page

+ 3, × 3: Numbers refer to  
Sensitivity

20  
15-φ 5  
10 (%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No BH-15N

4 OF 5

METRIC

G.W.P. 312-99-00 LOCATION N 5 135 831.26 E 318 356.01 ORIGINATED BY SL  
 HWY 69 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 15.05.04 - 16.05.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
							20 40 60 80 100	20 40 60 80 100	20 40 60					
			16	SS	8		195							
							194							
							193							
			2	TW	PH		192						17.0	C <sub>u</sub> =0.63 e <sub>u</sub> =1.432 C <sub>sw</sub> =0.26 C <sub>v</sub> =62m <sup>2</sup> /y G=2.80
							191							
							190							
			17	SS	WH		189							
							188							
							187							
							186							
			18	SS	10									

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15 10 5  
(%) STRAIN AT FAILURE

ONTMT4 8415.GPJ 21/07/04

## RECORD OF BOREHOLE No BH-15N

5 OF 5

METRIC

G.W.P. 312-99-00

LOCATION N 5 135 831.26 E 318 356.01

ORIGINATED BY SL

HWY 69

BOREHOLE TYPE NW Casing

COMPILED BY WM

DATUM Geodetic

DATE 15.05.04 - 16.05.04

CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							
183.8							185								
41.8	SILT, some clay, trace sand Compact Grey (ML)		3	TW	PH		184								
							183								
							182								
							181								
	some sand		19	SS	24		180								
							179								
							178								
							177								
176.4			20	SS	71/ 225										
49.2	END OF BOREHOLE AT 49.15 m. BOREHOLE GROUTED WITH BENTONITE SLURRY TO SURFACE.														

ONTMT4 8415.GPJ 21/07/04

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity20  
15  
10  
(%) STRAIN AT FAILURE

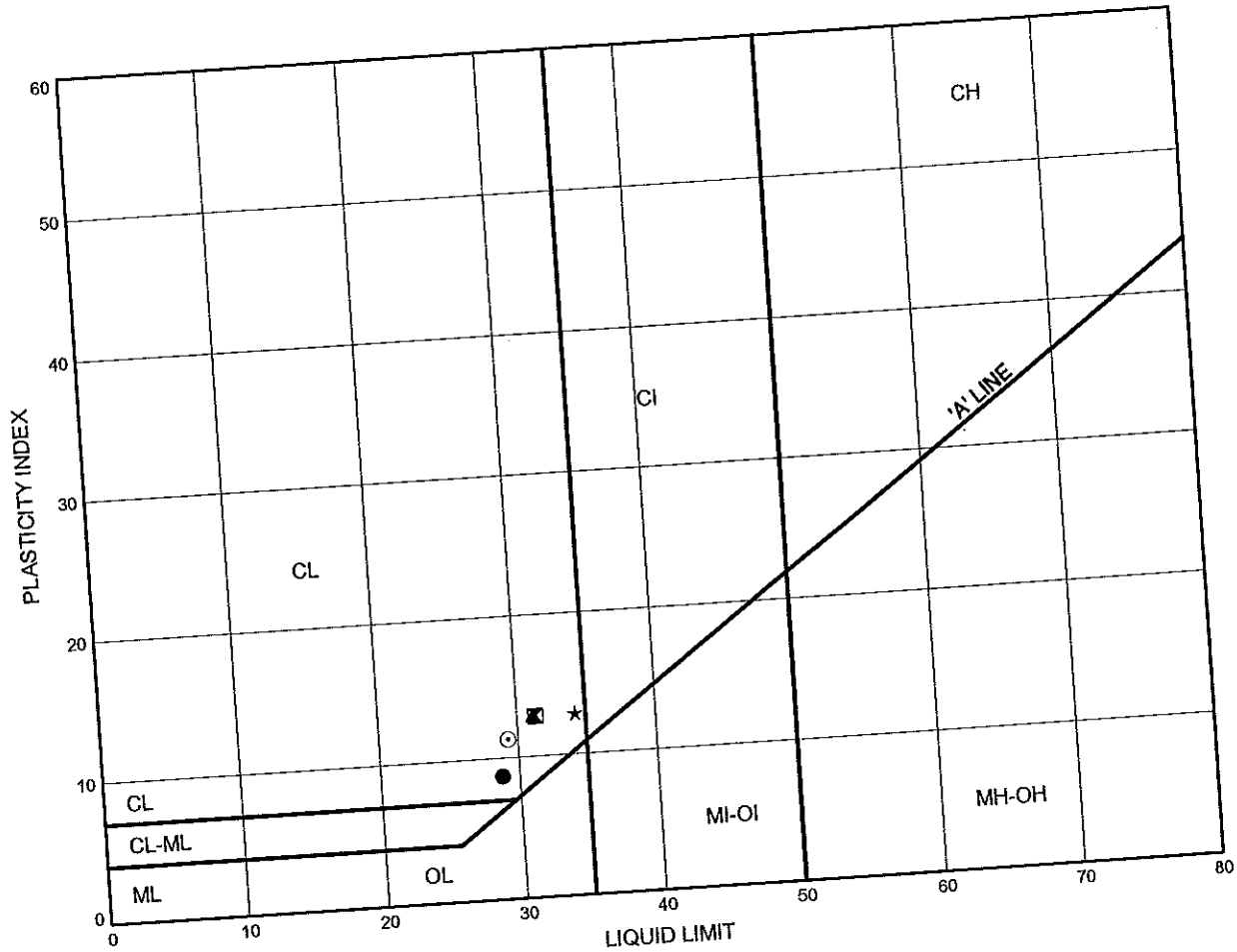
## **Appendix B**

### **Laboratory Test Results**

# HWY 69 Four Laning, Estaire ATTERBERG LIMITS TEST RESULTS

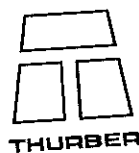
FIGURE B1

SBL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-10S	2.90	216.30
⊠	BH-12S	13.41	205.86
▲	BH-13S	11.89	207.25
★	BH-13S	33.22	185.92
⊙	BH-14S	7.32	211.71

Date July 2004  
Project 312-99-00

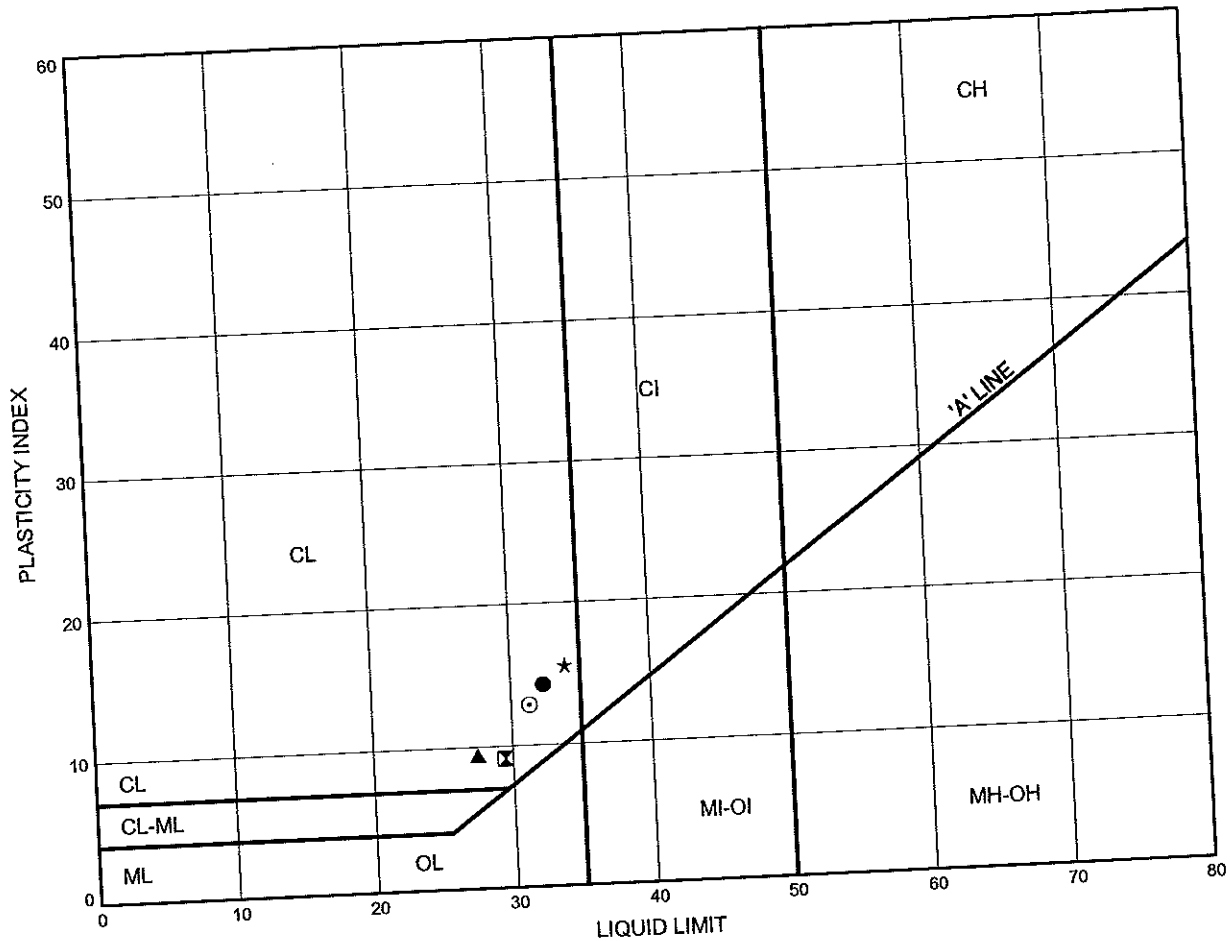


Prep'd WM  
Chkd. PJB

# HWY 69 Four Laning, Estaire ATTERBERG LIMITS TEST RESULTS

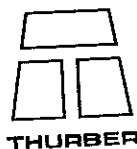
FIGURE B2

SBL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-14S	11.89	207.14
⊠	BH-14S	33.22	185.81
▲	BH-15S	14.94	208.50
★	BH-16S	14.94	208.49
⊙	BH-18S	16.99	207.44

Date July 2004  
Project 312-99-00

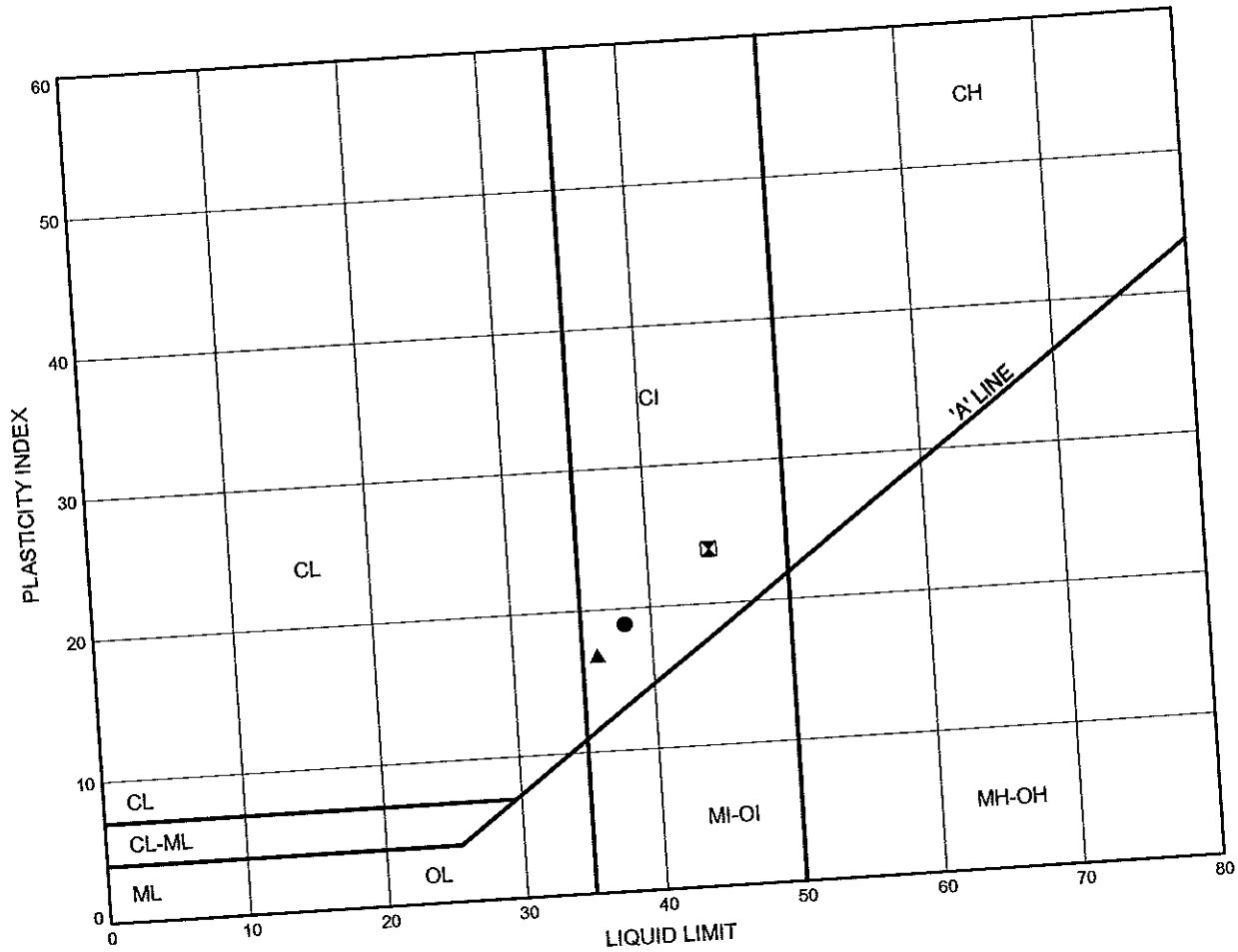


Prep'd WM  
Chkd. PJB

# HWY 69 Four Laning, Estaire ATTERBERG LIMITS TEST RESULTS

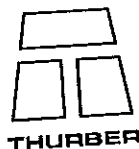
FIGURE B3

SBL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-11S	10.36	208.81
⊠	BH-12S	7.32	211.95
▲	BH-17S	16.45	207.64

Date July 2004  
Project 312-99-00



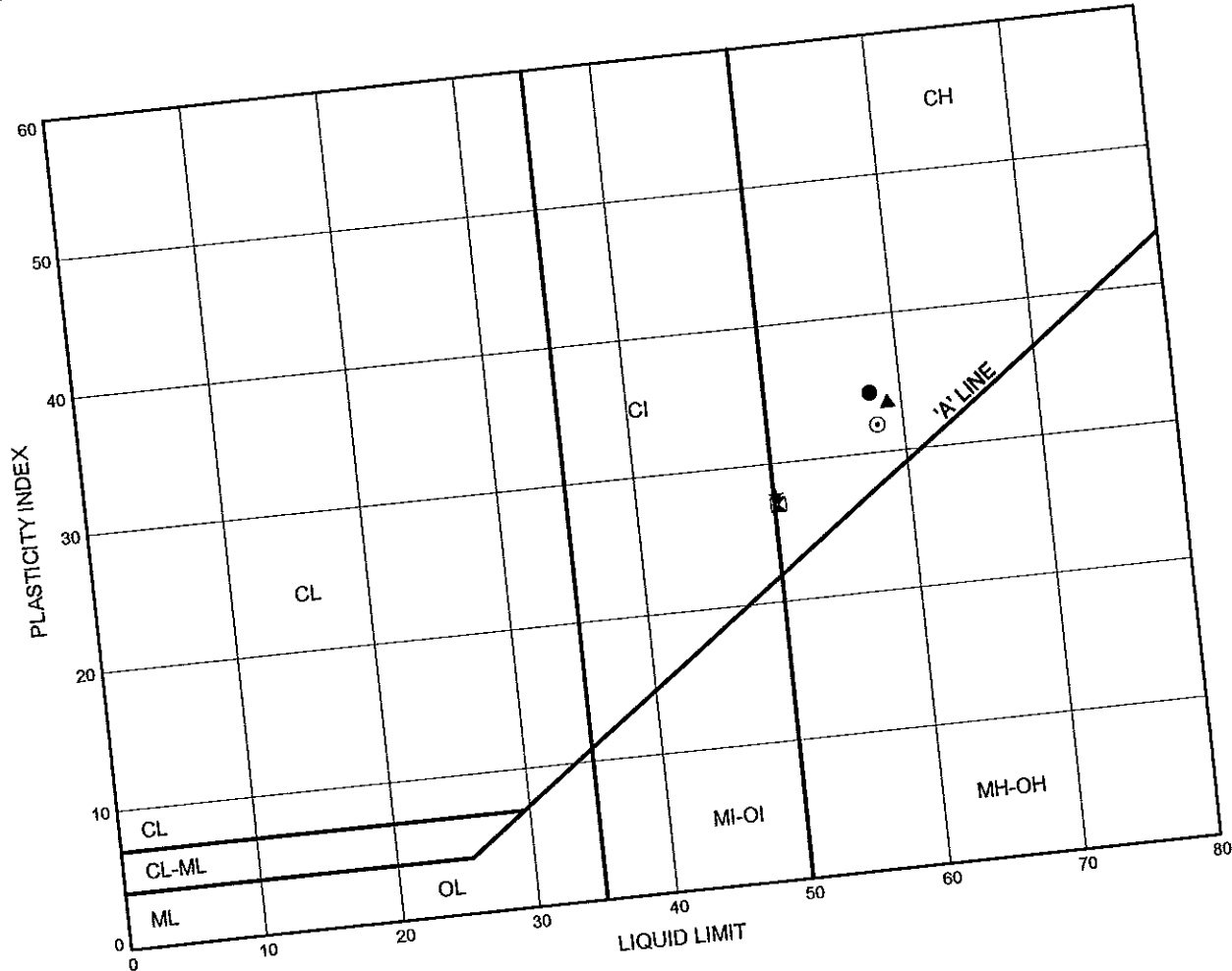
THURBER

Prep'd WM  
Chkd. PJB

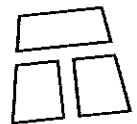
# HWY 69 Four Laning, Estaire ATTERBERG LIMITS TEST RESULTS

FIGURE B4

SBL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-11S	14.94	204.23
⊠	BH-13S	21.03	198.11
▲	BH-14S	24.08	194.95
★	BH-15S	27.12	196.32
⊙	BH-15S	36.27	187.17



THURBER

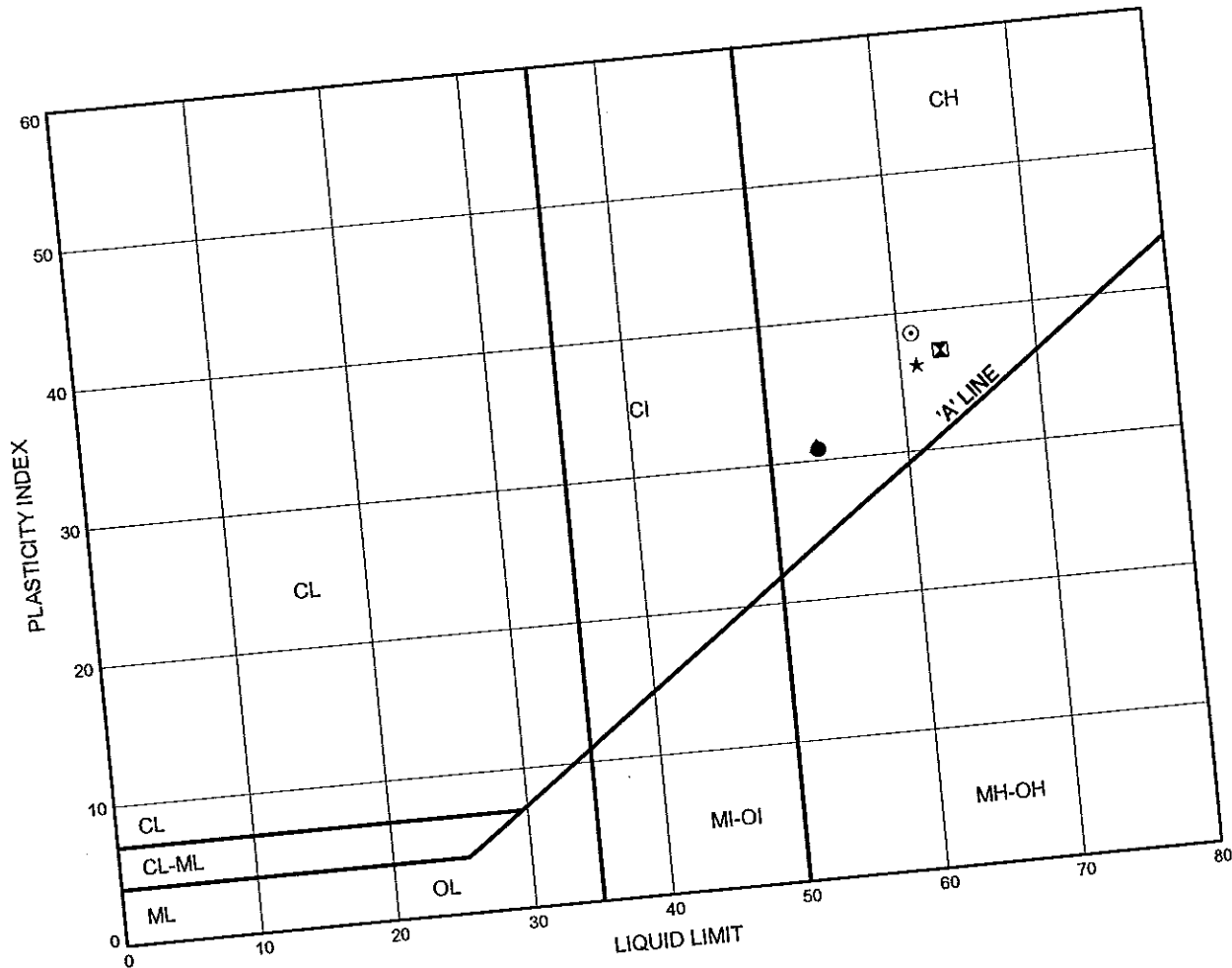
Date July 2004  
Project 312-99-00

Prep'd WM  
Chkd. PJB

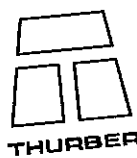
# HWY 69 Four Laning, Estaire ATTERBERG LIMITS TEST RESULTS

FIGURE B5

SBL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-16S	27.13	196.30
⊠	BH-16S	33.22	190.21
▲	BH-17S	30.18	193.91
★	BH-17S	36.27	187.82
⊙	BH-18S	29.18	195.25



THURBER

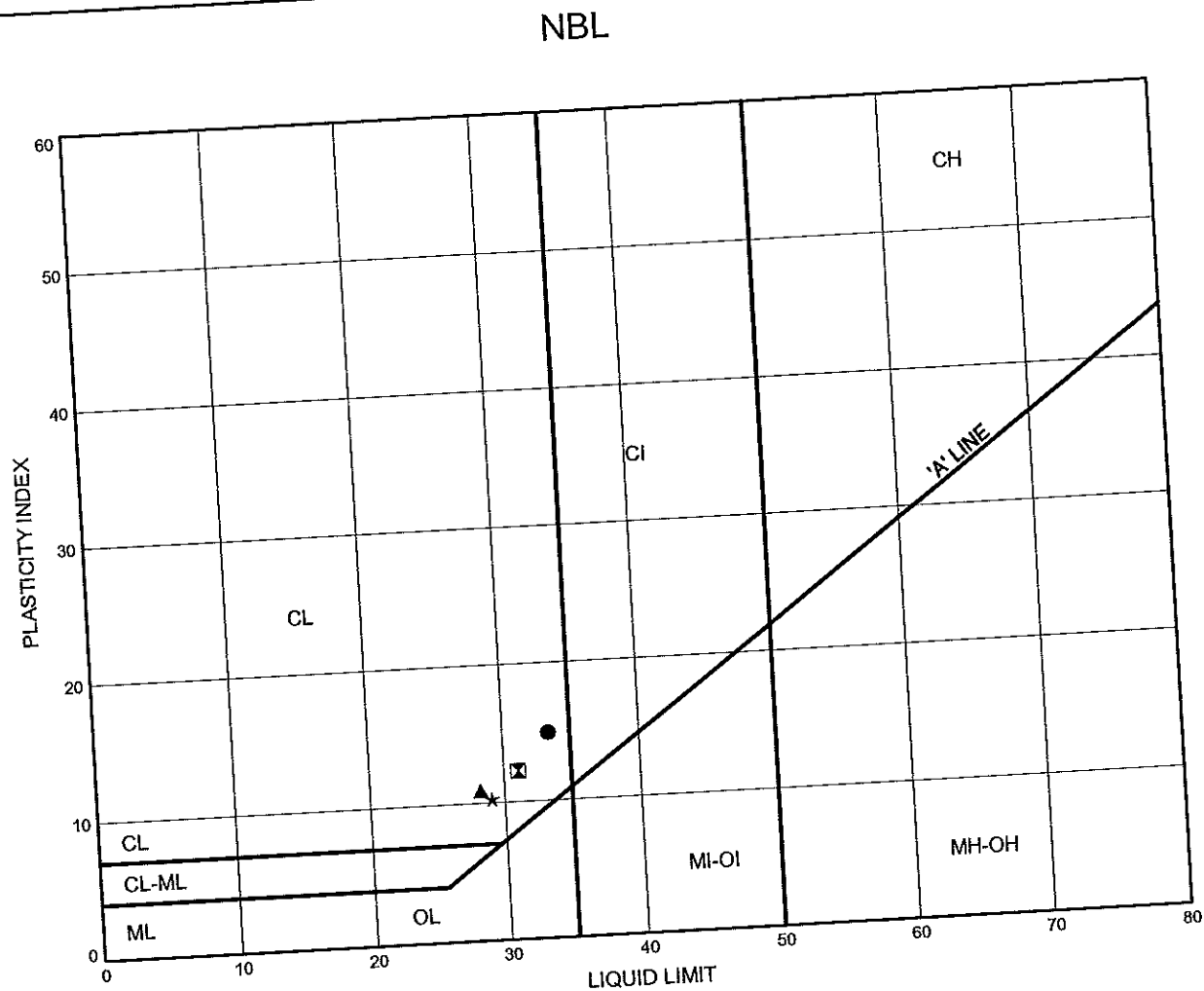
Date July 2004  
Project 312-99-00

Prep'd WM  
Chkd. PJB



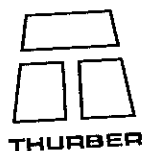
# HWY 69 Four Laning, Estaire ATTERBERG LIMITS TEST RESULTS

FIGURE B6



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-10N	11.89	208.36
⊠	BH-12N	15.54	208.66
▲	BH-14N	17.07	207.25
★	BH-15N	17.07	208.48

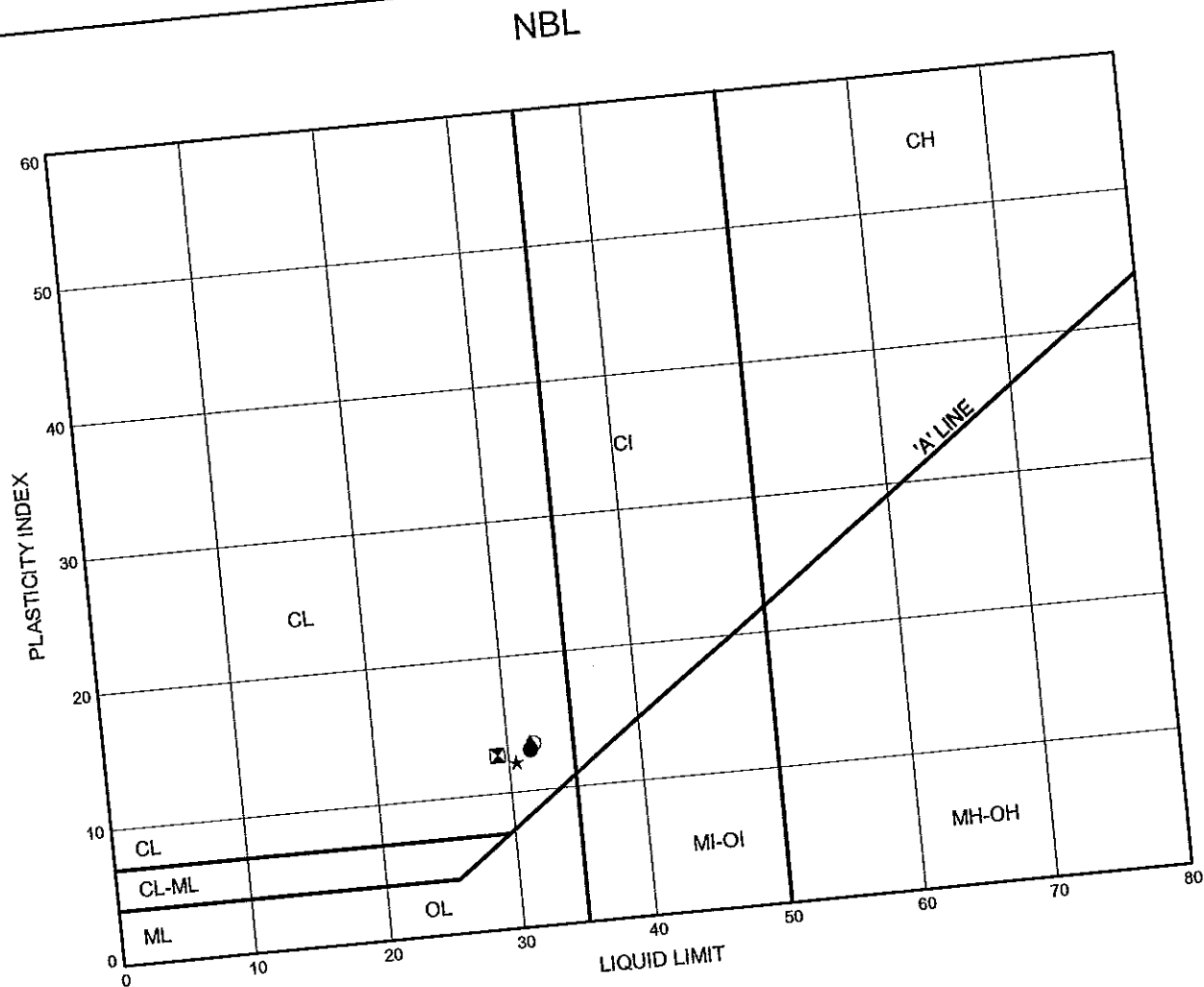
Date July 2004  
 Project 312-99-00



Prep'd WM  
 Chkd. PJB

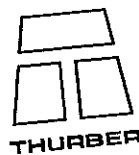
# HWY 69 Four Laning, Estaire ATTERBERG LIMITS TEST RESULTS

FIGURE B7



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-15N	21.64	203.91
■	BH-6N	4.27	214.73
▲	BH-7N	2.74	216.50
★	BH-8N	7.92	211.31
⊙	BH-9N	8.84	210.32

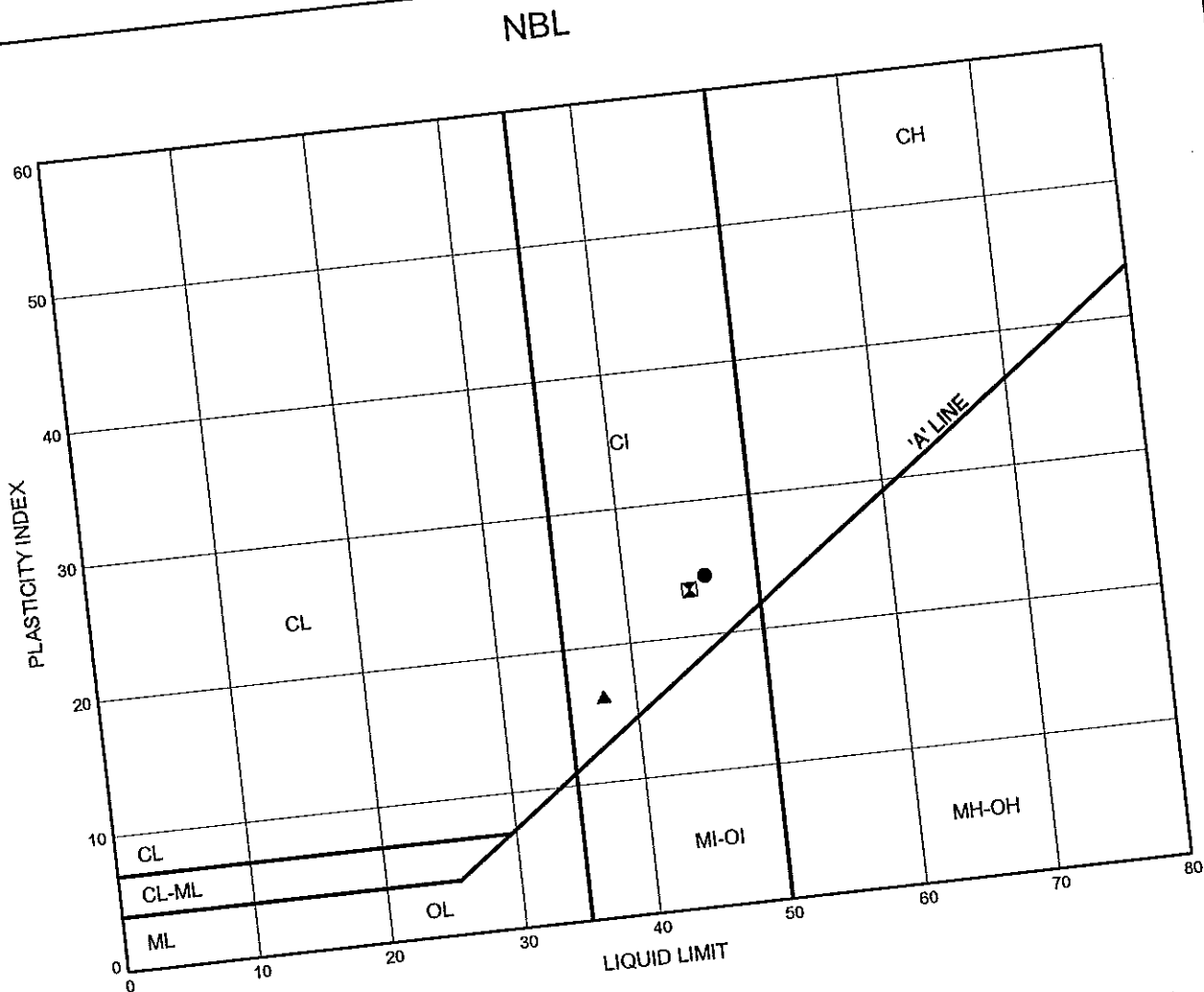
Date July 2004  
 Project 312-99-00



Prep'd WM  
 Chkd. PJB

# HWY 69 Four Laning, Estaire ATTERBERG LIMITS TEST RESULTS

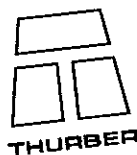
FIGURE B8



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-12N	24.08	200.12
⊠	BH-15N	27.74	197.81
▲	BH-7N	7.92	211.32

Date July 2004

Project 312-99-00

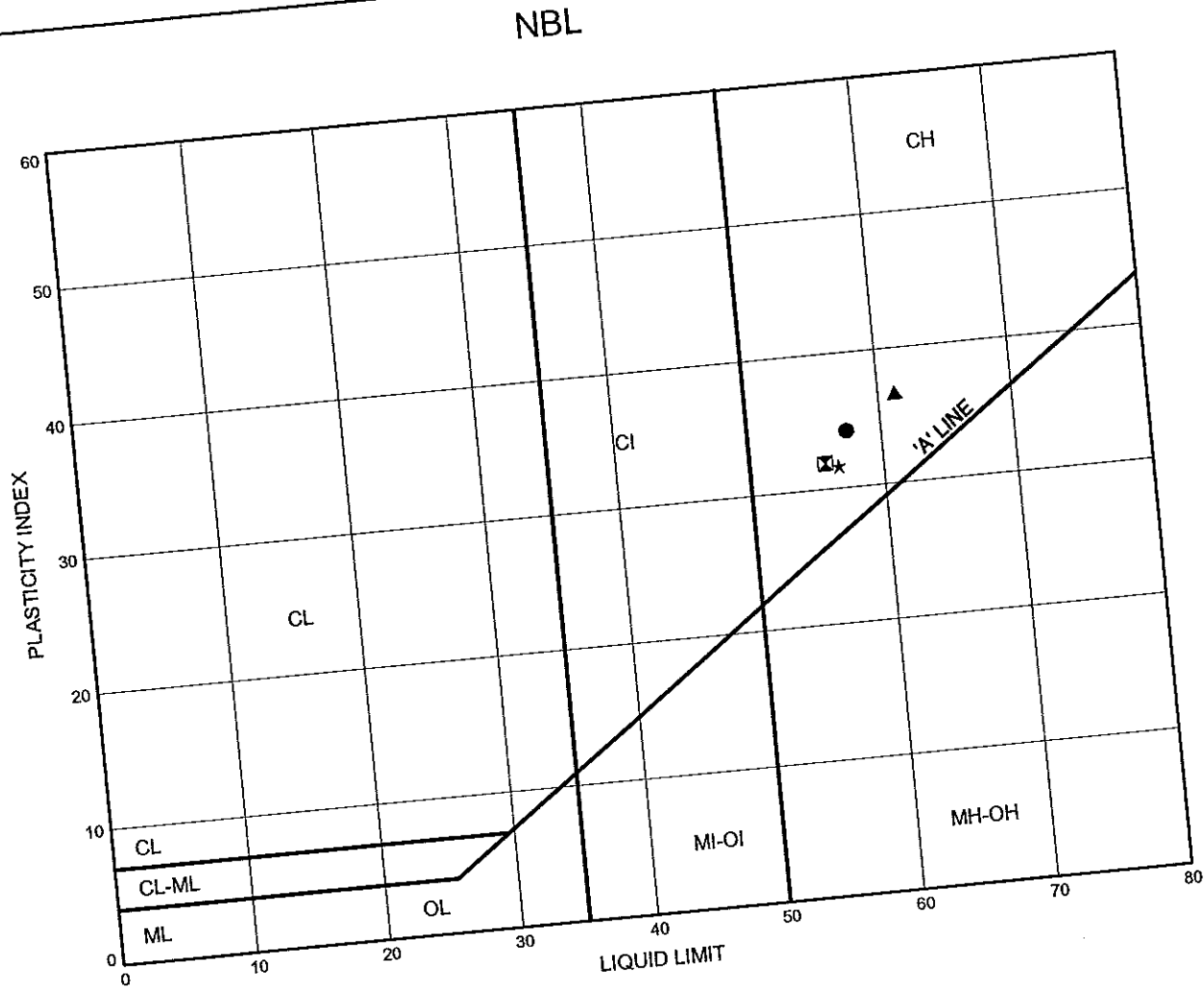


Prep'd WM

Chkd. PJB

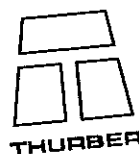
# HWY 69 Four Laning, Estaire ATTERBERG LIMITS TEST RESULTS

FIGURE B9



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-10N	24.08	196.17
⊠	BH-11N	33.22	187.74
▲	BH-12N	33.22	190.98
★	BH-13N	30.18	193.82

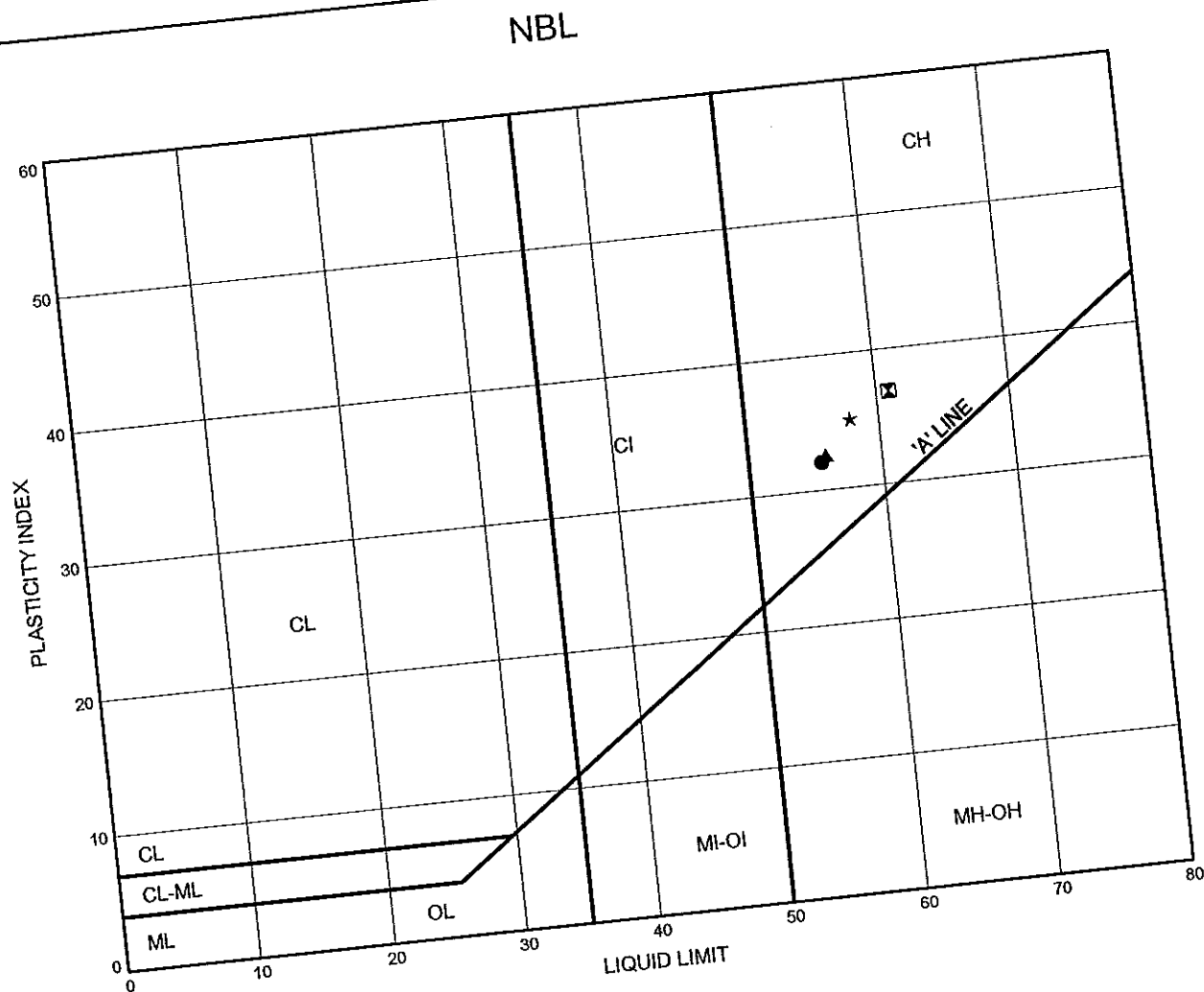
Date July 2004  
 Project 312-99-00



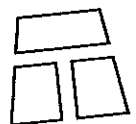
Prep'd WM  
 Chkd. PJB

# HWY 69 Four Laning, Estaire ATTERBERG LIMITS TEST RESULTS

FIGURE B10



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-13N	36.27	187.73
⊠	BH-14N	28.50	195.82
▲	BH-8N	15.54	203.69
★	BH-9N	17.98	201.18



THURBER

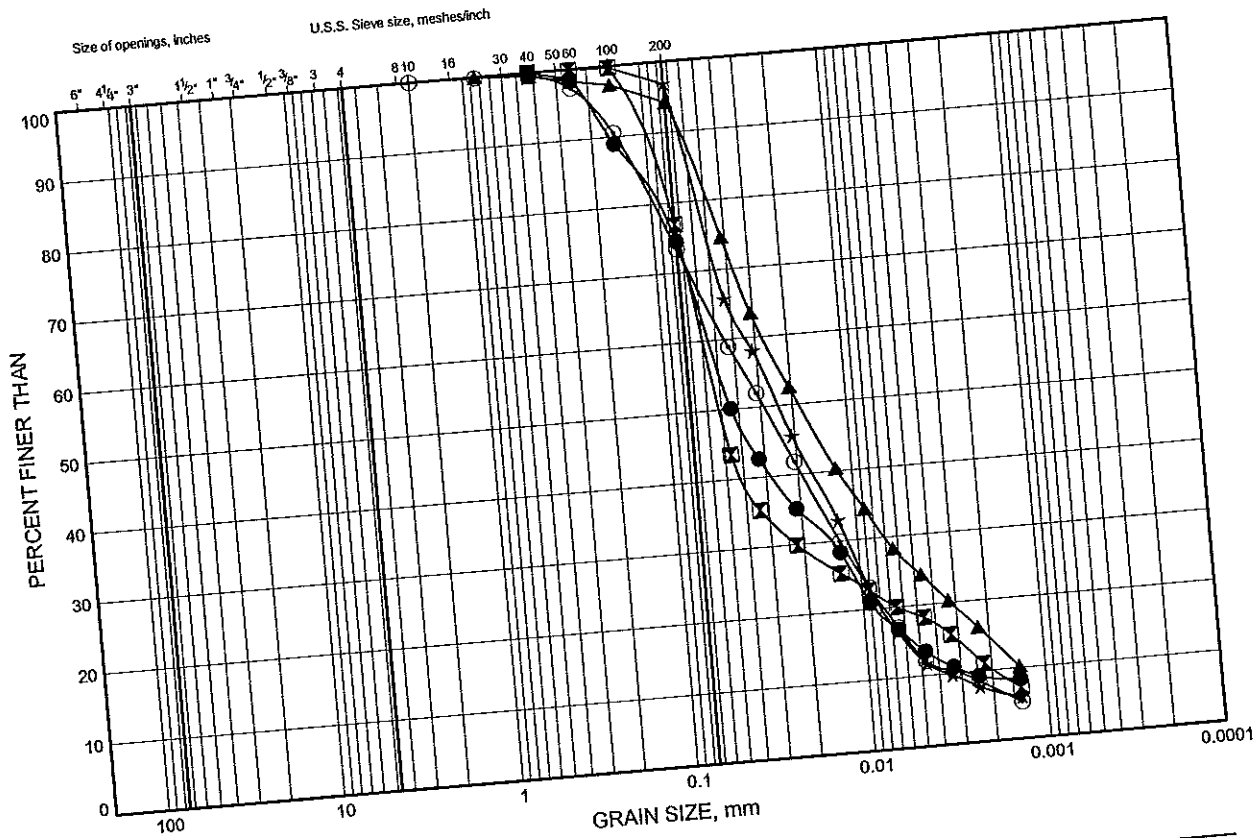
Prep'd ..... WM  
Chkd. .... PJB

Date July 2004  
Project 312-99-00

# HWY 69 Four Laning, Estaire GRAIN SIZE DISTRIBUTION

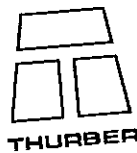
FIGURE B11

SBL - SAND/SILT



COBBLE SIZE	GRAVEL		SAND			FINE GRAINED
	COARSE	FINE	COARSE	MEDIUM	FINE	

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-16S	7.32	216.11
⊠	BH-18S	3.35	221.08
▲	BH-18S	6.40	218.03
★	BH-6S	1.83	217.16
⊙	BH-7S	2.74	216.31



THURBER

Date July 2004  
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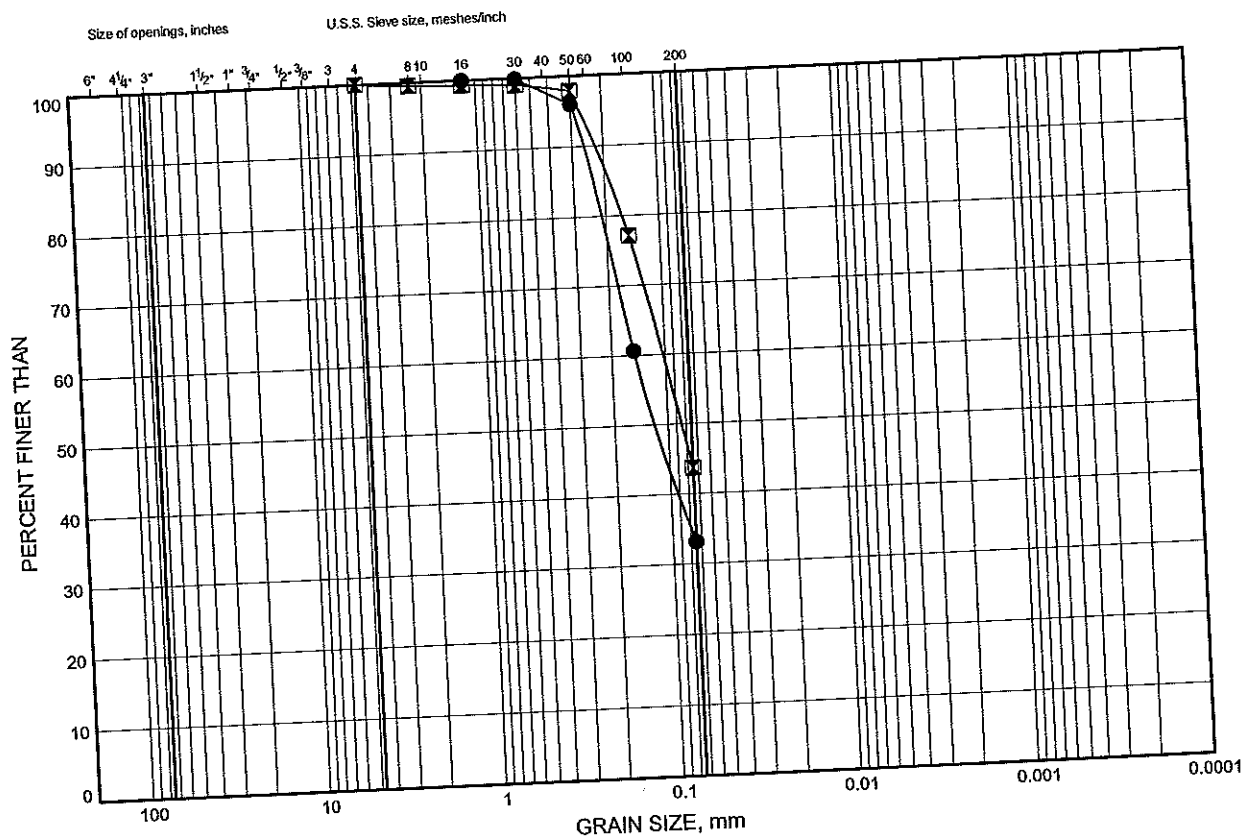
Prep'd WM  
Chkd. PJB

# HWY 69 Four Laning, Estaire

## GRAIN SIZE DISTRIBUTION

FIGURE B12

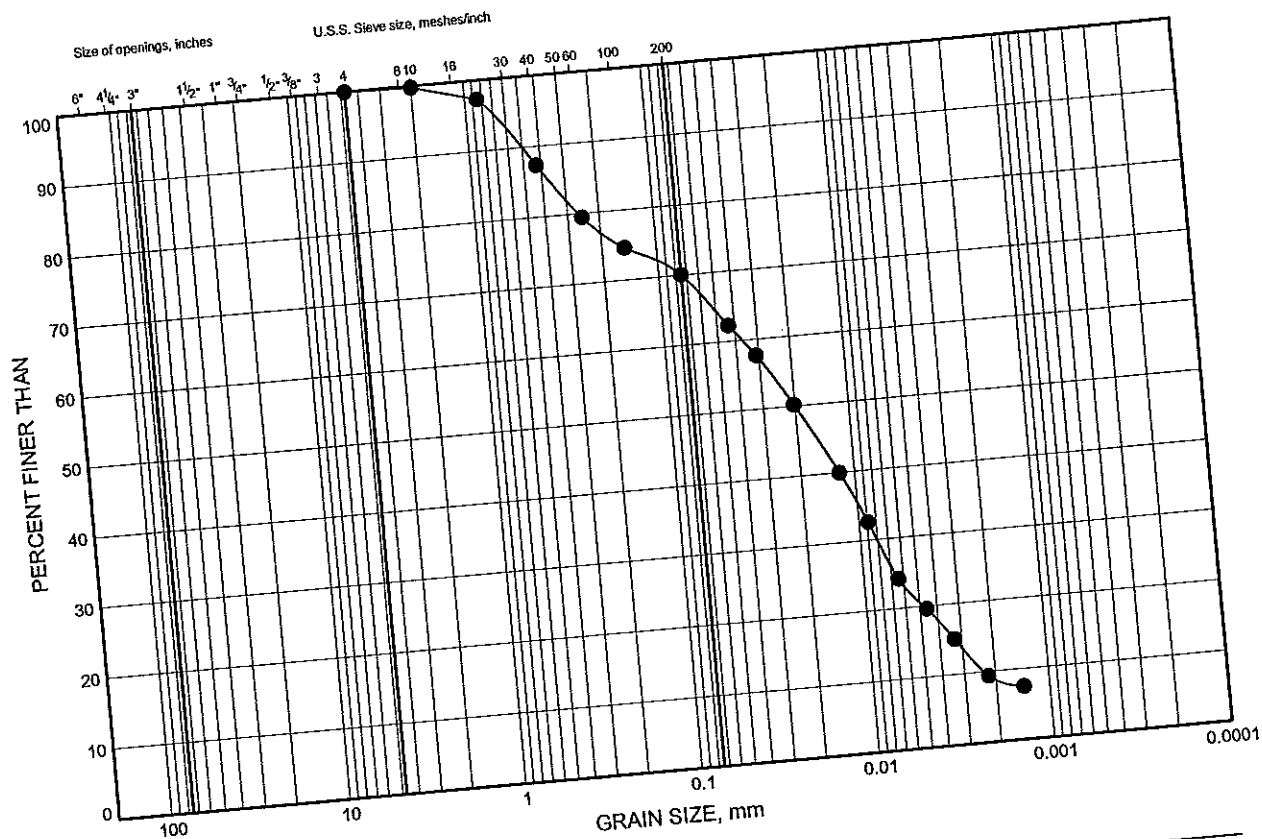
### SBL - SAND/SILT



# HWY 69 Four Laning, Estaire GRAIN SIZE DISTRIBUTION

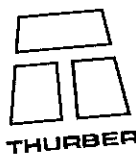
FIGURE B13

SBL - SANDY SILT



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-17S	48.46	175.63



THURBER

Date July 2004  
Project 312-99-00

Prep'd WM  
Chkd. PJB

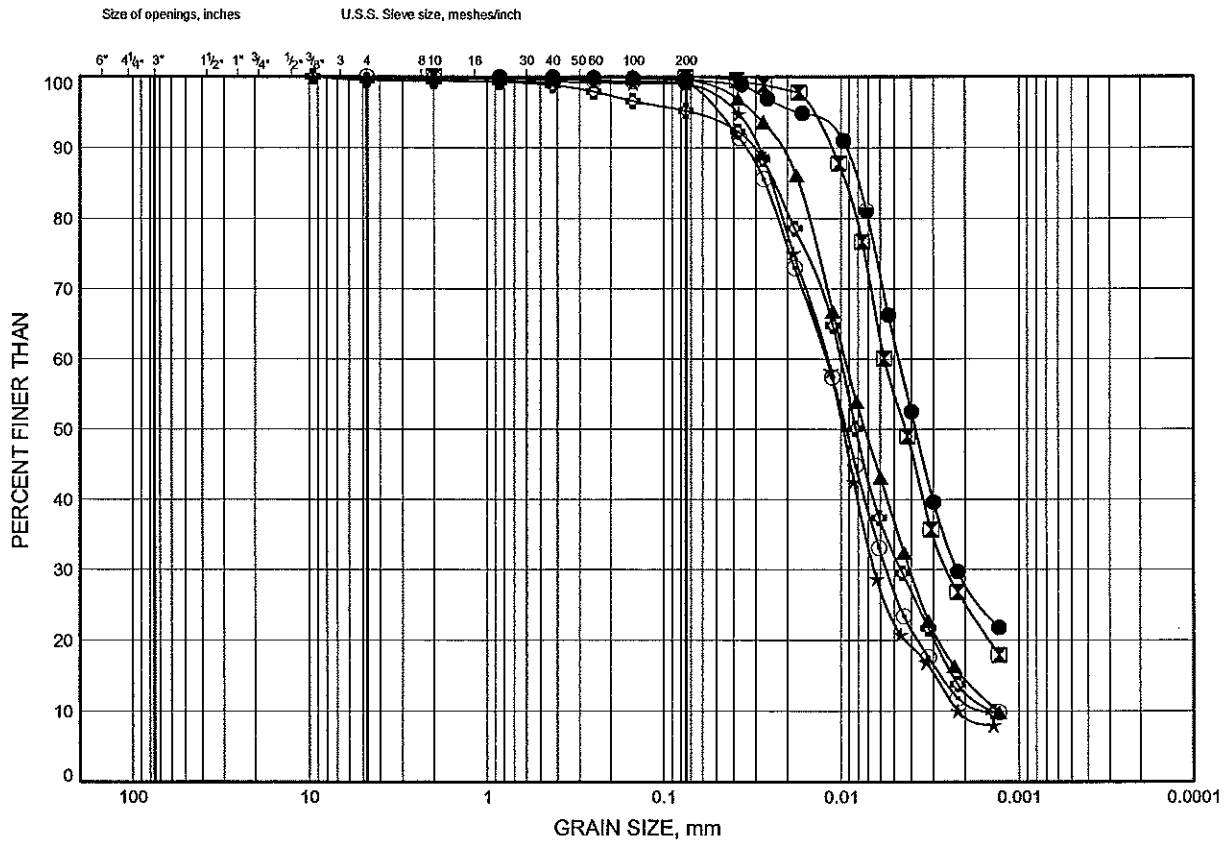


# HWY 69 Four Laning, Estaire

## GRAIN SIZE DISTRIBUTION

FIGURE B14

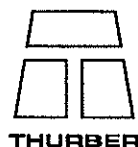
### SBL - SILT (ML) / CLAYEY SILT (CL-ML)



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-11S	16.46	202.71
⊠	BH-15S	39.32	184.12
▲	BH-15S	45.42	178.02
★	BH-16S	42.37	181.06
⊙	BH-17S	42.37	181.72
⊕	BH-18S	46.02	178.41

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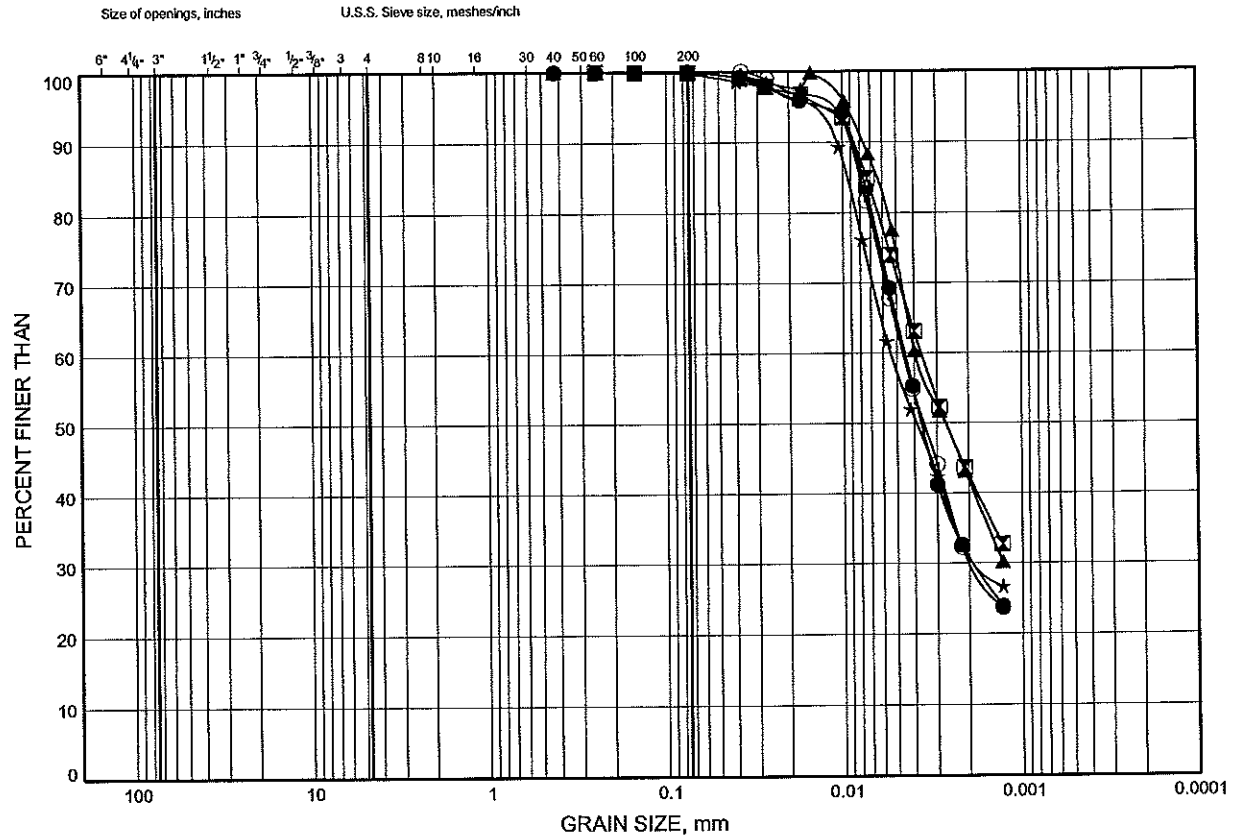
Prep'd WM

Chkd. PJB

# HWY 69 Four Laning, Estaire GRAIN SIZE DISTRIBUTION

FIGURE B15

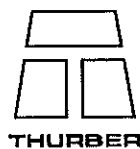
### SBL - SILTY CLAY (CL)



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-10S	2.90	216.30
⊠	BH-12S	13.41	205.86
▲	BH-13S	33.22	185.92
★	BH-14S	7.32	211.71
⊙	BH-14S	33.22	185.81

Date July 2004  
Project 312-99-00

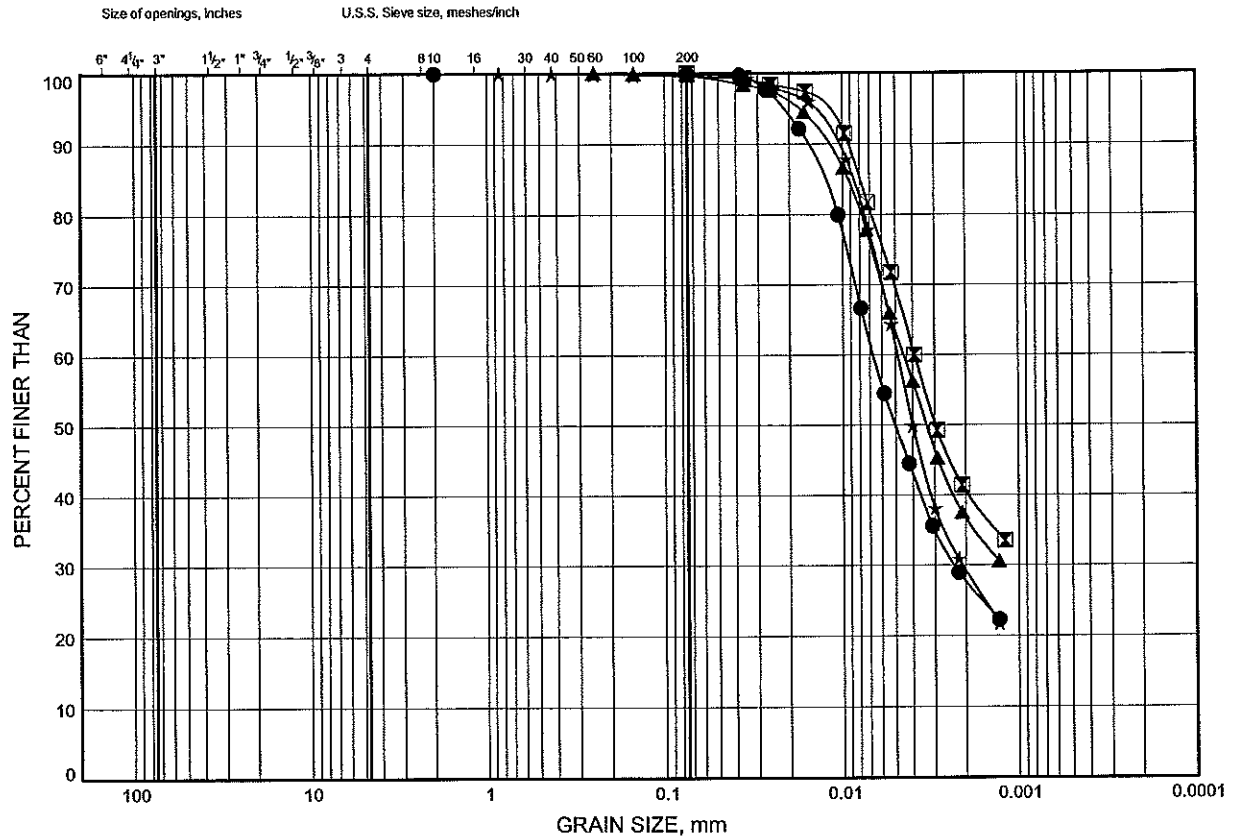


Prep'd WM  
Chkd. PJB

# HWY 69 Four Laning, Estaire GRAIN SIZE DISTRIBUTION

FIGURE B16

### SBL - SILTY CLAY (CL)

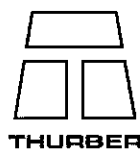


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-15S	14.94	208.50
⊠	BH-16S	14.94	208.49
▲	BH-18S	16.99	207.44
★	BH-9S	2.74	216.50

Date July 2004

Project 312-99-00



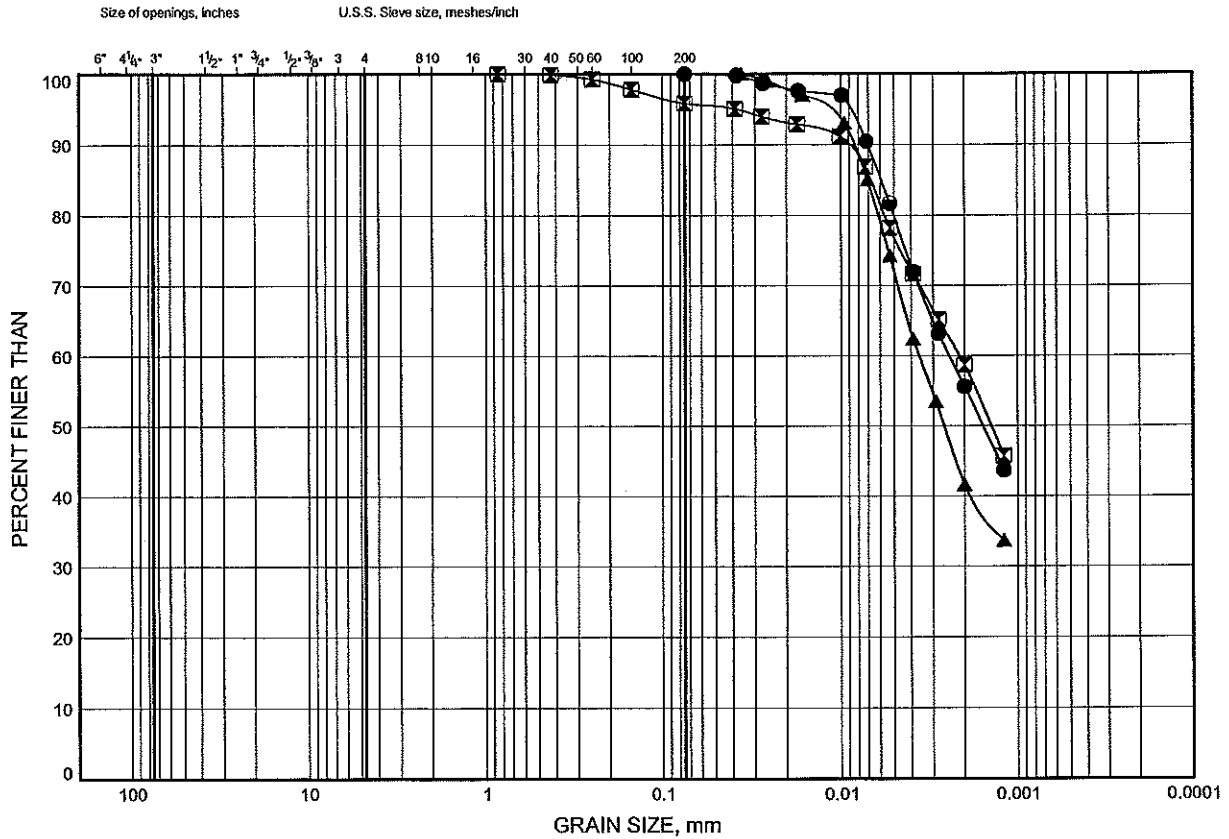
Prep'd WM

Chkd. PJB

# HWY 69 Four Laning, Estaire GRAIN SIZE DISTRIBUTION

FIGURE B17

### SBL - SILTY CLAY (CI)

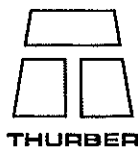


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-11S	10.36	208.81
⊠	BH-12S	7.32	211.95
▲	BH-17S	16.45	207.64

Date July 2004

Project 312-99-00



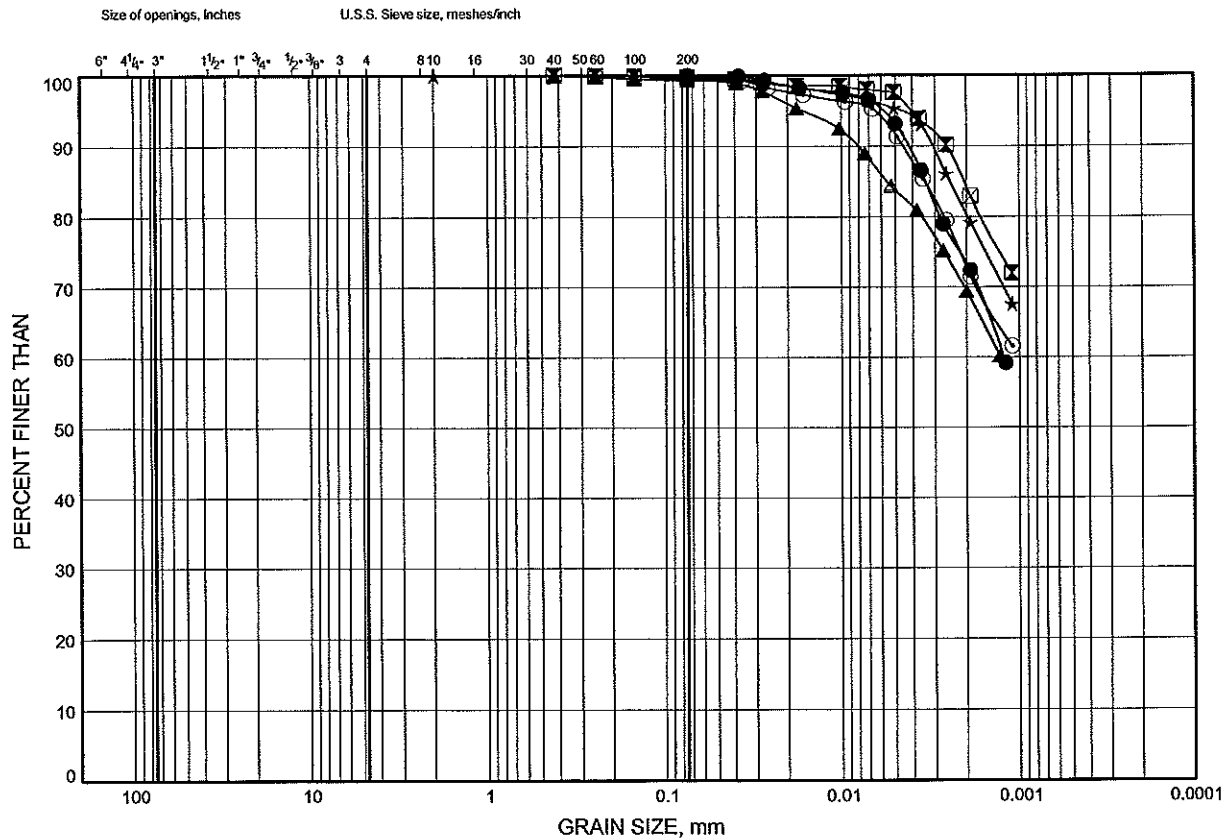
Prep'd WM

Chkd. PJB

# HWY 69 Four Laning, Estaire GRAIN SIZE DISTRIBUTION

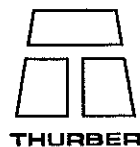
FIGURE B18

### SBL - SILTY CLAY (CH)



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-11S	14.94	204.23
⊠	BH-14S	24.08	194.95
▲	BH-15S	27.12	196.32
★	BH-15S	36.27	187.17
⊙	BH-16S	27.13	196.30

Date July 2004  
Project 312-99-00

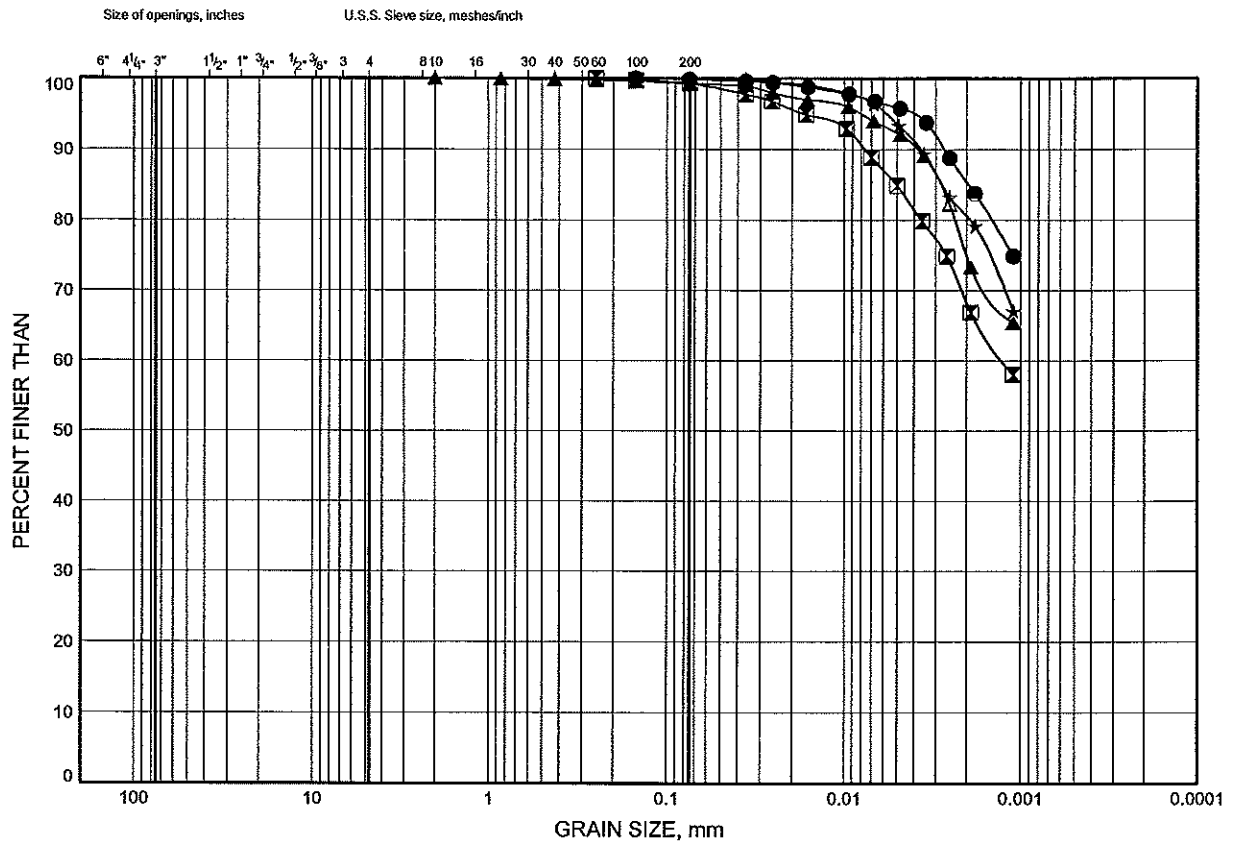


Prep'd WM  
Chkd. PJB

# HWY 69 Four Laning, Estaire GRAIN SIZE DISTRIBUTION

FIGURE B19

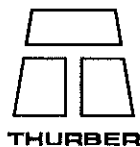
### SBL - SILTY CLAY (CH)



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-16S	33.22	190.21
⊠	BH-17S	30.18	193.91
▲	BH-17S	36.27	187.82
★	BH-18S	29.18	195.25

Date July 2004  
Project 312-99-00

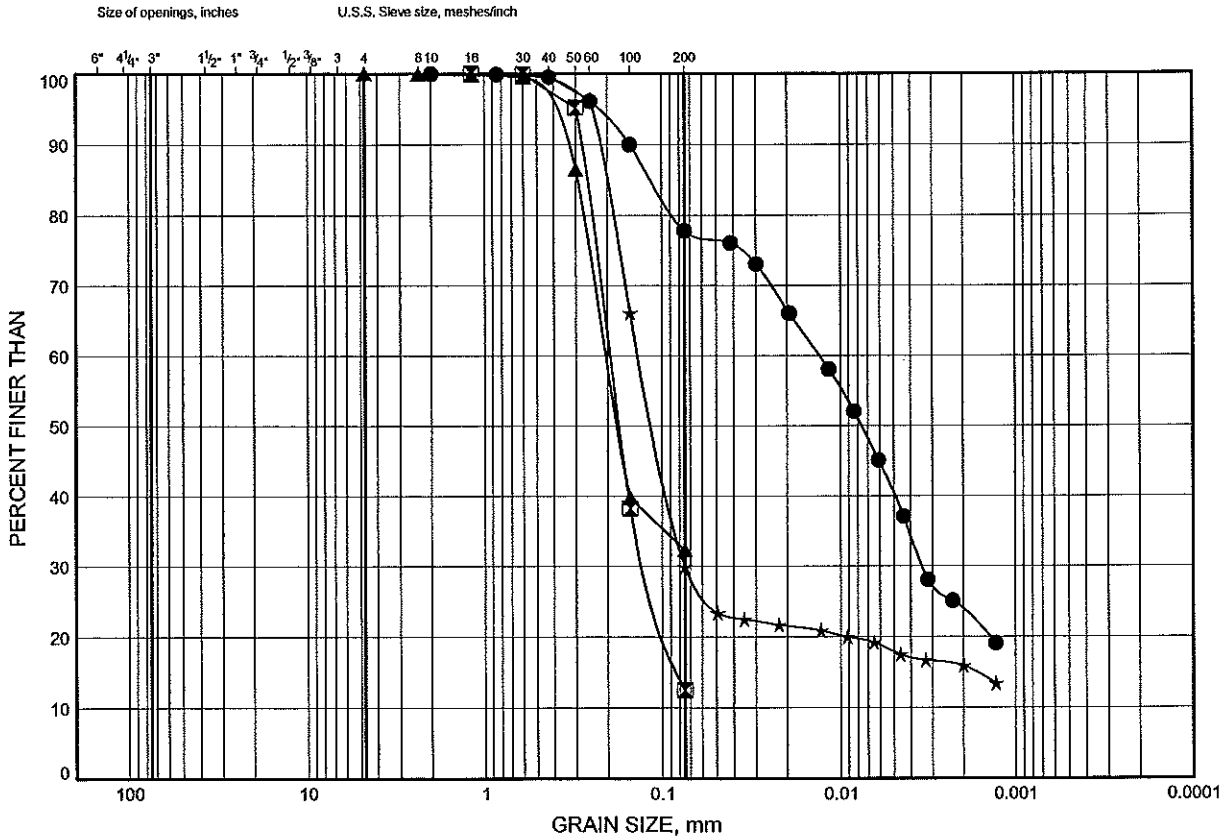


Prep'd WM  
Chkd. PJB

# HWY 69 Four Laning, Estaire GRAIN SIZE DISTRIBUTION

FIGURE B20

### NBL - SAND / SILT

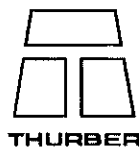


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-10N	2.74	217.51
⊠	BH-10N	5.79	214.46
▲	BH-11N	4.27	216.69
★	BH-11N	7.32	213.64

Date July 2004

Project 312-99-00



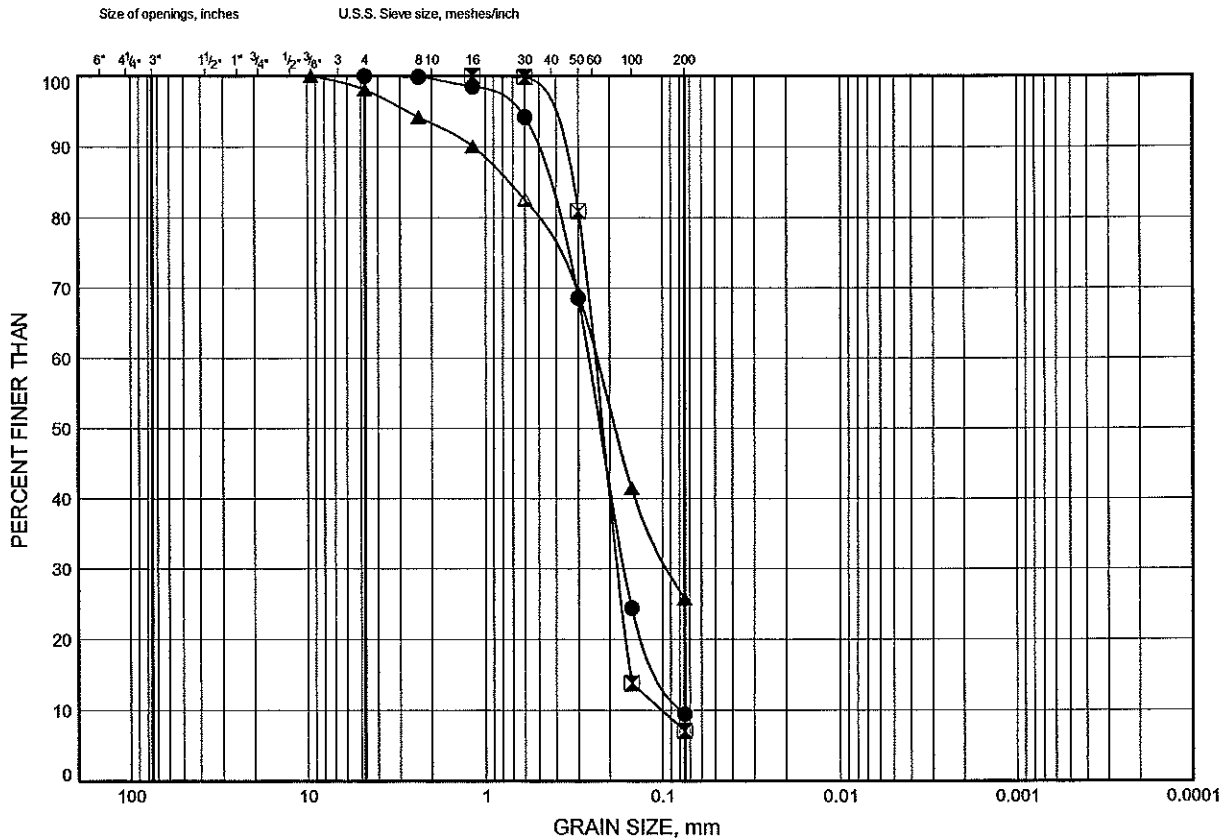
Prep'd WM

Chkd. PJB

# HWY 69 Four Laning, Estaire GRAIN SIZE DISTRIBUTION

FIGURE B21

## NBL - SAND / SILT

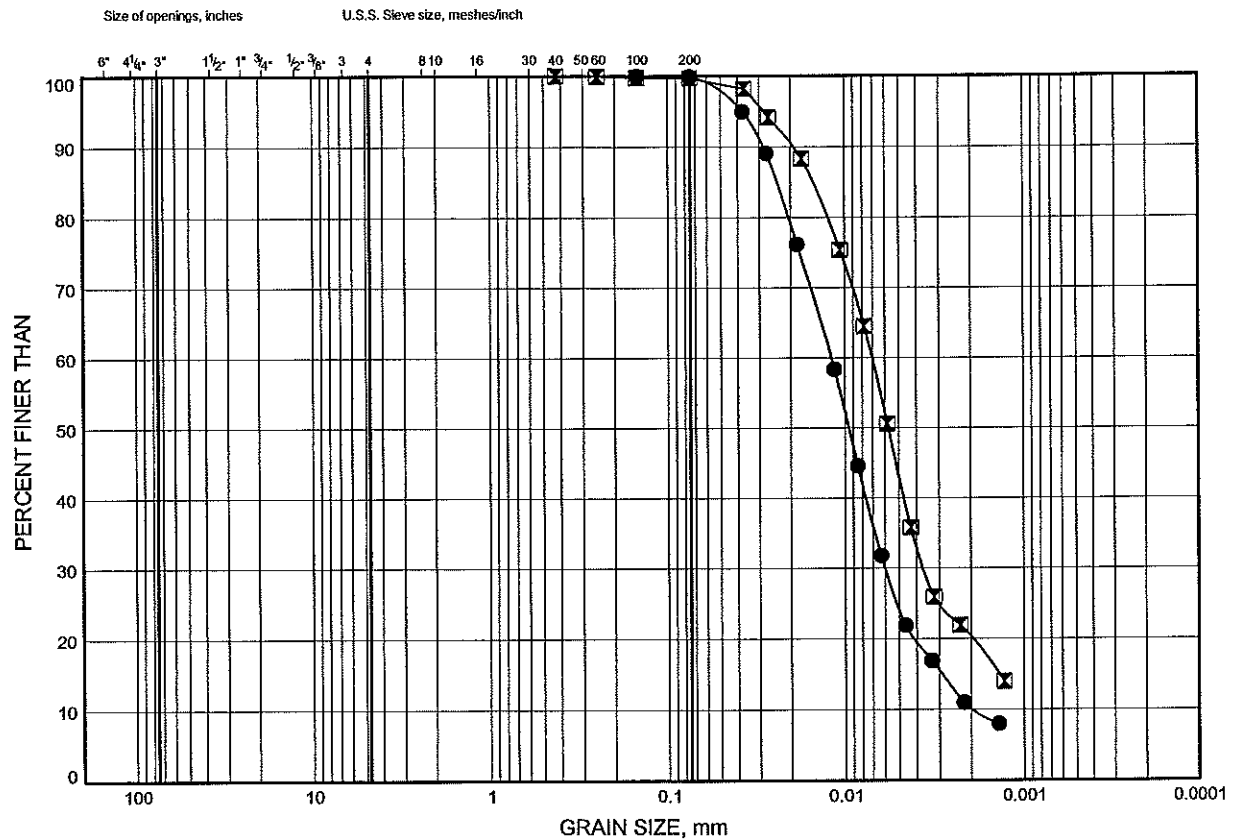




# HWY 69 Four Laning, Estaire GRAIN SIZE DISTRIBUTION

FIGURE B22

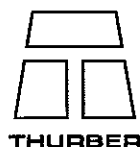
### NBL - SILT



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-12N	42.37	181.83
⊠	BH-14N	39.93	184.39

Date July 2004  
Project 312-99-00

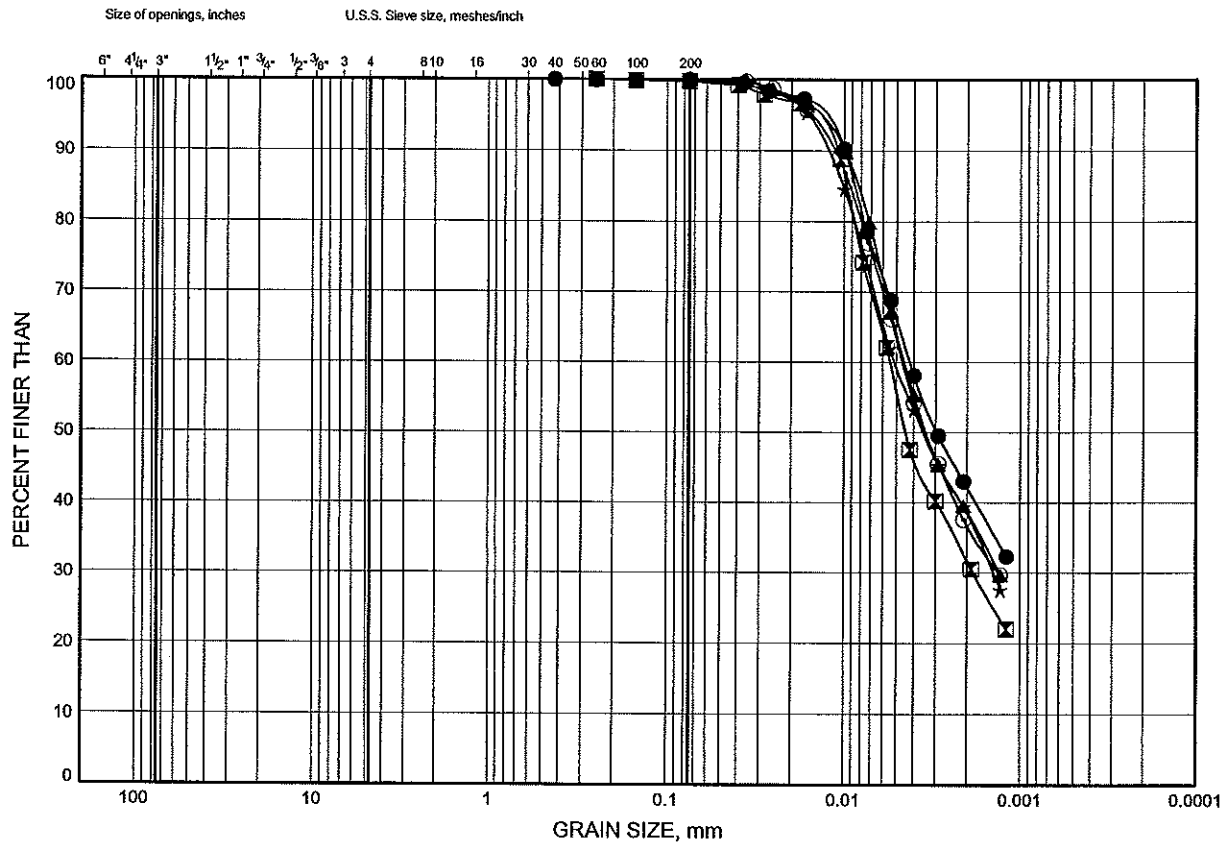


Prep'd WM  
Chkd. PJB

# HWY 69 Four Laning, Estaire GRAIN SIZE DISTRIBUTION

FIGURE B23

### NBL - SILTY CLAY (CL)



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-10N	11.89	208.36
⊠	BH-11N	8.84	212.12
▲	BH-12N	15.54	208.66
★	BH-14N	17.07	207.25
⊙	BH-15N	17.07	208.48

Date July 2004  
Project 312-99-00

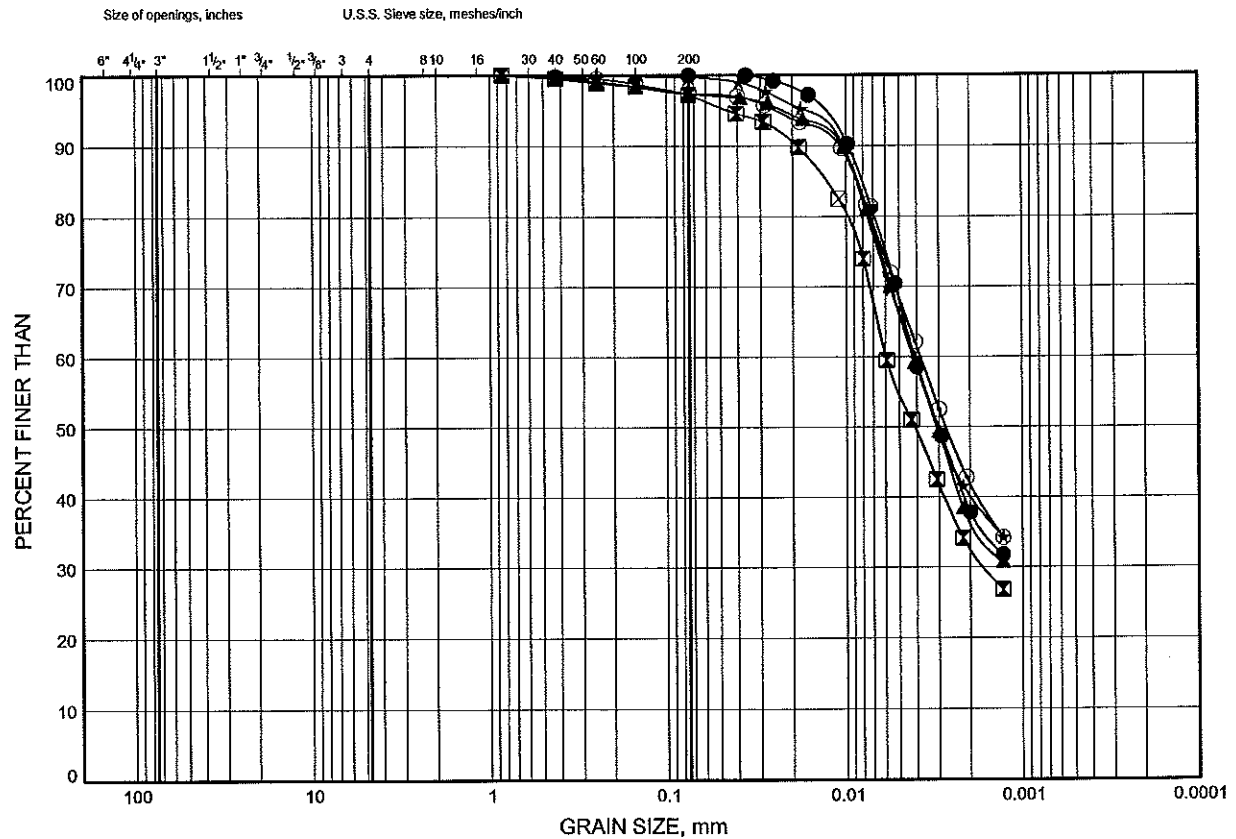


Prep'd WM  
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# HWY 69 Four Laning, Estaire GRAIN SIZE DISTRIBUTION

FIGURE B24

## NBL - SILTY CLAY (CL)

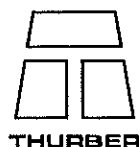


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-15N	21.64	203.91
⊠	BH-6N	4.27	214.73
▲	BH-7N	2.74	216.50
★	BH-8N	7.92	211.31
⊙	BH-9N	8.84	210.32

Date July 2004

Project 312-99-00



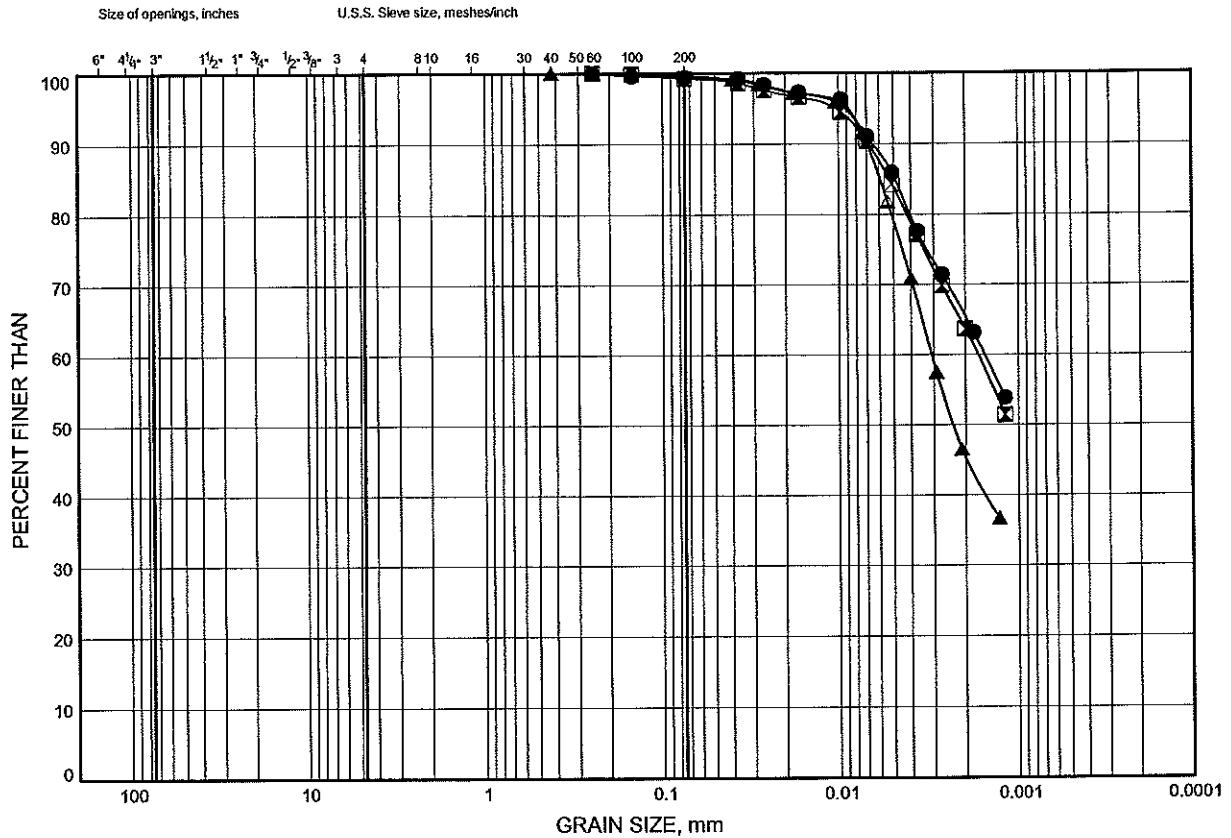
Prep'd WM

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# HWY 69 Four Laning, Estaire GRAIN SIZE DISTRIBUTION

FIGURE B25

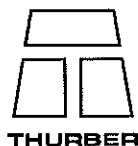
NBL - SILTY CLAY (CI)



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-12N	24.08	200.12
⊠	BH-15N	27.74	197.81
▲	BH-7N	7.92	211.32

Date July 2004  
Project 312-99-00

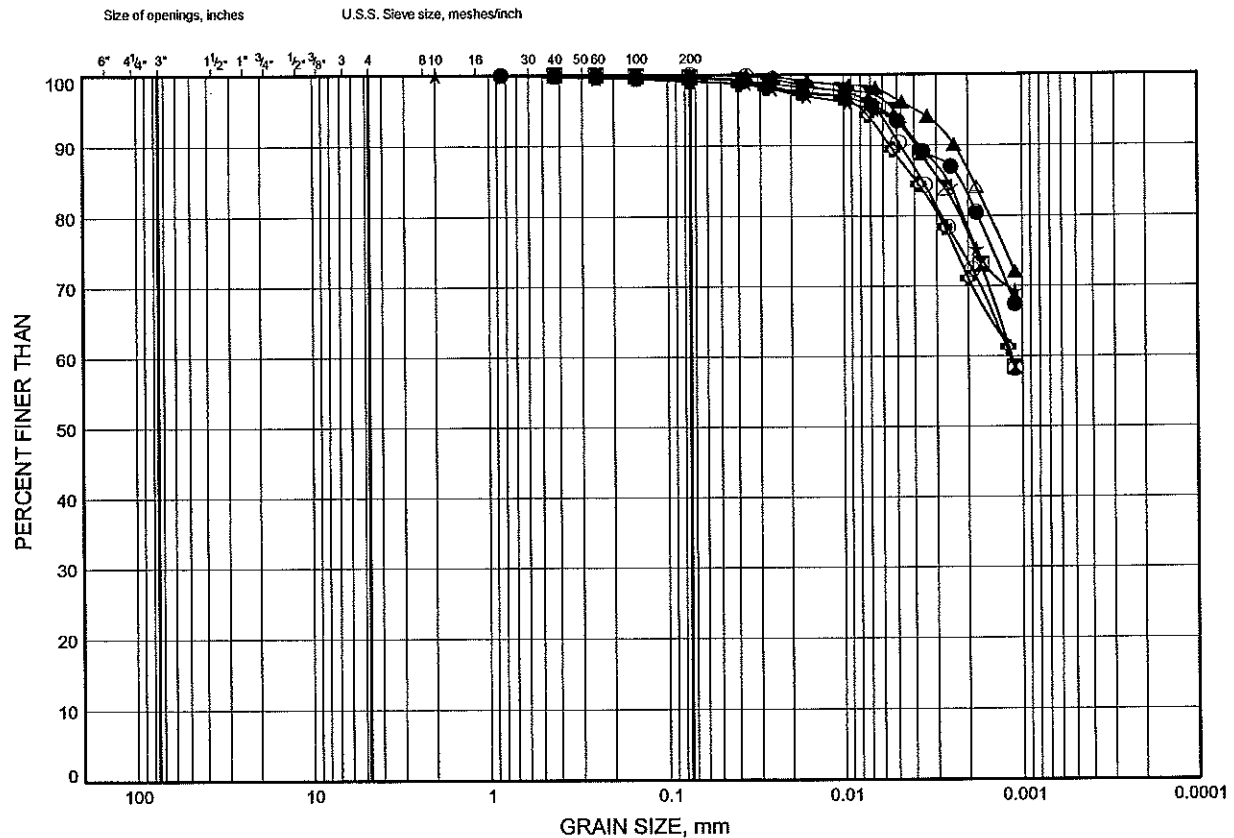


Prep'd WM  
Chkd. PJB

# HWY 69 Four Laning, Estaire GRAIN SIZE DISTRIBUTION

FIGURE B26

### NBL - SILTY CLAY (CH)



**Highway 69 Four Laning (SBL), Estaire  
Point Load Test Results**

Depth			Is50	UCS (MPa)
feet	Inches	m		
BH-6S				
10	1	3.07	10.52	252.49
11	0	3.35	10.43	250.41
12	0	3.66	9.91	237.89
13	0	3.96	7.04	169.02
13	8	4.17	10.78	258.75
14	8	4.47	14.52	348.48
15	8	4.78	6.96	166.94
16	8	5.08	11.74	281.71
17	8	5.38	10.09	242.06
18	8	4.78	6.26	150.24
19	2	4.93	10.78	258.75
20	2	5.23	3.91	93.90

Total Rock Core			
Average	Minimum	Maximum	
226	94	348	MPa
Run #	Average		
1	227.45		
2	259.59		
3	167.63		

Depth			Is50	UCS (MPa)
feet	Inches	m		
BH-10S				
169	8	51.71	10.87	260.84
170	3	51.89	4.26	102.25
170	8	52.02	5.22	125.20
171	8	52.32	4.00	95.99
173	0	52.73	11.96	286.92
175	0	53.34	8.87	212.85
176	2	53.70	10.35	248.32
177	10	54.20	7.30	175.28
178	10	54.51	6.17	148.16

Total Rock Core			
Average	Minimum	Maximum	
184	96	287	MPa
Run #	Average		
1	181.54		
2	110.60		
3	249.88		
4	190.59		

Note: Occasional hidden closed joints along rock core

Depth			Is50	UCS (MPa)
feet	Inches	m		
BH-11S				
57	0	17.37	12.61	302.57
57	6	17.53	12.43	298.40
58	6	17.83	7.30	175.28
59	2	18.03	6.35	152.33
60	4	18.39	6.96	166.94
61	4	18.69	8.17	196.15
62	8	19.10	10.35	248.32
63	8	19.41	5.13	123.12
66	8	20.32	7.74	185.72
67	8	20.62	1.65	39.65
68	0	20.73	3.04	73.04

Total Rock Core			
Average	Minimum	Maximum	
178	40	303	MPa
Run #	Average		
1	232.15		
2	181.54		
3	185.72		
4	185.72		
5	56.34		

**Highway 69 Four Laning (SBL), Estaire  
Point Load Test Results**

Depth			Is50	UCS (MPa)
feet	Inches	m		
BH-12S				
52	6	16.00	10.09	242.06
54	10	16.71	6.43	154.42
55	6	16.92	2.09	50.08
56	3	17.15	6.00	143.98
56	10	17.32	2.09	50.08
57	6	17.53	6.96	166.94
58	1	17.70	7.04	169.02
58	8	17.88	8.00	191.98
59	6	18.14	6.35	152.33
60	6	18.44	7.13	171.11
62	0	18.90	7.30	175.28

Total Rock Core			
Average	Minimum	Maximum	
141	50	192	MPa
Run #	Average		
1	242.06		
3	116.16		
4	144.51		
5	161.72		
6	175.28		

Note: Point load test at 16.92 m was performed on Diorite

Depth		Is50	UCS (MPa)	
feet	Inches			m
BH-13S				
124	4	37.90	0.52	12.52
124	9	38.02	4.78	114.77
125	5	38.23	5.00	119.99
126	6	38.56	5.30	127.29
127	2	38.76	5.13	123.12
128	6	39.17	6.96	166.94
129	1	39.34	8.17	196.15
130	2	39.67	9.74	233.71
130	8	39.83	5.83	139.81
131	2	39.98	7.65	183.63
132	2	40.28	5.13	123.12
133	2	40.59	6.04	145.03
134	0	40.84	5.56	133.55

Total Rock Core			
Average	Minimum	Maximum	
140	13	234	MPa
Run #	Average		
1	82.43		
2	164.50		
3	146.33		

Depth		m	Is50	UCS (MPa)
feet	Inches			
BH-14S				
122	6	37.34	14.35	344.31
123	6	37.64	13.22	317.18
124	2	37.85	10.17	244.15
124	10	38.05	3.56	85.56
125	4	38.20	11.39	273.36
126	6	38.56	8.26	198.24
127	4	38.81	1.39	33.39
128	2	39.07	8.17	196.15
129	2	39.37	6.61	158.59

Total Rock Core			
Average	Minimum	Maximum	
170	33	273	MPa
Run #	Average		
1	301.88		
2	179.46		
3	146.59		

**Highway 69 Four Laning (SBL), Estaire  
Point Load Test Results**

Depth			Is50	UCS (MPa)
feet	Inches	m		
BH-15S				
169	10	51.77	5.22	125.20
172	1	52.45	11.13	267.10
173	6	52.88	5.74	137.72
174	6	53.19	2.96	70.95
175	7	53.52	1.57	37.56
175	6	53.49	6.69	160.68
178	7	54.43	2.52	60.51
179	5	54.69	4.78	114.77

Total Rock Core			
Average	Minimum	Maximum	
122	38	267	MPa
Run #	Average		
1	125.20		
2	267.10		
3	82.08		
4	111.99		

Depth			Is50	UCS (MPa)
feet	Inches	m		
BH-16S				
164	8	50.19	10.78	258.75
167	4	51.00	4.78	114.77
168	5	51.33	0.78	18.78
169	4	51.61	8.69	208.67
170	2	51.87	7.13	171.11
170	10	52.07	11.22	269.19
171	11	52.40	3.04	73.04
173	1	52.76	12.00	287.97
173	11	53.01	6.87	164.85
174	11	53.31	5.13	123.12
176	0	53.64	10.87	260.84
176	7	53.82	8.69	208.67
177	3	54.03	10.26	246.23

Total Rock Core			
Average	Minimum	Maximum	
185	19	288	MPa
Run #	Average		
1	258.75		
2	128.33		
3	183.63		
4	238.58		

Note: Point load test at 50.19 m was performed on boulder  
 Point load tests at 51.33 and 52.40 m were performed on hidden joint



### Highway 69 Four Laning (NBL), Estaire Point Load Test Results

Depth			Is50	UCS (MPa)
feet	inches	m		
BH-6N				
35	5	10.80	13.56	325.53
38	5	11.71	3.65	87.64
39	6	12.04	11.56	277.53
40	8	12.40	13.56	325.53
41	3	12.57	7.83	187.80
42	0	12.80	7.65	183.63
43	10	13.36	6.35	152.33

Note: Insufficient sample for point load test in Run #2  
Occasional hidden closed joints along rock core

Total Rock Core			
Average	Minimum	Maximum	
220	88	326	MPa
Run #	Average		
1	230.23		
3	325.53		
4	187.80		
5	167.98		

Depth			Is50	UCS (MPa)
feet	Inches	m		
BH-7N				
33	3	10.13	2.70	64.69
35	2	10.72	6.43	154.42
35	8	10.87	9.04	217.02
36	2	11.02	8.17	196.15
36	7	11.16	3.48	83.47
37	2	11.33	8.87	212.85
38	8	11.79	7.83	187.80
39	2	11.94	11.04	265.01
41	0	12.50	13.69	328.66
41	10	12.75	6.00	143.98
42	8	13.00	8.96	214.93

Note: Occasional hidden closed joints along rock core

Total Rock Core			
Average	Minimum	Maximum	
188	65	329	MPa
Run #	Average		
1	64.69		
2	189.20		
3	187.28		
4	229.19		

Depth			Is50	UCS (MPa)
feet	Inches	m		
BH-8N				
73	2	22.30	7.30	175.28
75	1	22.89	4.26	102.25
75	4	22.96	4.43	106.42
75	9	23.09	4.00	95.99
77	2	23.52	8.78	210.76
78	2	23.83	9.74	233.71
79	4	23.88	1.57	37.56
80	0	24.08	2.43	58.43
82	0	24.38	2.26	54.25
83	0	24.99	6.43	154.42
84	5	25.73	9.74	233.71

Note: Point load test at 22.30 m was performed on boulder  
Point load tests at 22.96 and 23.88m were performed on Diorite  
Point load tests at 23.88, 24.08 & 24.38 m were performed at hidden joint  
Occasional hidden closed joints along rock core

Total Rock Core			
Average	Minimum	Maximum	
133	38	234	MPa
Run #	Average		
1	175.28		
2	101.55		
3			
4	125.20		

**Highway 69 Four Laning (NBL), Estaire  
Point Load Test Results**

Depth			Is50	UCS (MPa)
feet	Inches	m		
BH-9N				
70	4	21.44	4.78	114.77
71	4	21.74	4.00	95.99
72	4	22.05	5.13	123.12
73	6	22.40	1.74	41.73
75	0	22.86	8.87	212.85
75	8	23.06	8.78	210.76
76	4	23.27	3.83	91.82
77	4	23.57	8.87	212.85
78	4	23.88	12.43	298.40
78	0	23.77	10.52	252.49
79	4	24.18	3.04	73.04

Total Rock Core			
Average	Minimum	Maximum	
157	42	298	MPa
Run #	Average		
1	93.90		
2	171.81		
3	209.19		

Note: Point load test at 22.4 m was performed at hidden joint

Depth			Is50	UCS (MPa)
feet	Inches	m		
BH-10N				
140	3	42.75	10.35	248.32
140	6	42.82	3.91	93.90
141	4	43.08	0.52	12.52
143	0	43.59	7.22	173.20
143	8	43.79	6.00	143.98
144	0	43.89	6.87	164.85
144	6	44.04	8.00	191.98

Total Rock Core			
Average	Minimum	Maximum	
147	13	248	MPa
Run #	Average		
1	123.64		
2	191.98		

Note: Point load test at 43.08 m was performed at hidden joint

Depth		Is50	UCS (MPa)	
feet	inches			
BH-11N				
138	8	42.27	3.39	81.38
139	8	42.57	9.04	217.02
140	6	42.82	5.91	141.90
142	6	43.43	5.04	121.03
143	0	43.59	2.17	52.17
147	6	44.96	8.09	194.07
148	4	45.21	7.56	181.54

Total Rock Core			
Average	Minimum	Maximum	
141	52	217	MPa
Run #	Average		
1	149.20		
2	105.03		
3	187.80		

**Highway 69 Four Laning (NBL), Estaire  
Point Load Test Results**

Depth			Is50	UCS (MPa)
feet	Inches	m		
BH-12N				
158	0	48.16	8.96	214.93
159	3	48.54	9.48	227.45
159	11	48.74	6.52	156.50
161	6	49.23	0.43	10.43
162	1	49.40	12.69	304.66
162	10	49.63	8.35	200.33
163	6	49.83	13.13	315.09
164	6	50.14	6.69	160.68
165	6	50.44	9.22	221.19
166	11	50.88	0.09	2.09
168	6	51.36	4.43	106.42
170	7	51.99	2.52	60.51

Total Rock Core			
Average	Minimum	Maximum	
165	2	315	MPa
Run #	Average		
1	214.93		
2	131.46		
3	252.49		
4	232.32		
5	56.34		

Note: Point load test at 49.23 and 50.88 m were performed at hidden closed joints

Depth			Is50	UCS (MPa)
feet	inches	m		
BH-13N				
151	9	46.25	5.48	131.46
152	8	46.53	5.65	135.64
153	8	46.84	6.69	160.68
158	0	48.16	6.87	164.85
159	0	48.46	9.30	223.28
159	10	48.72	3.74	89.73
160	6	48.92	1.83	43.82
161	6	49.23	7.30	175.28
163	6	49.83	4.35	104.34

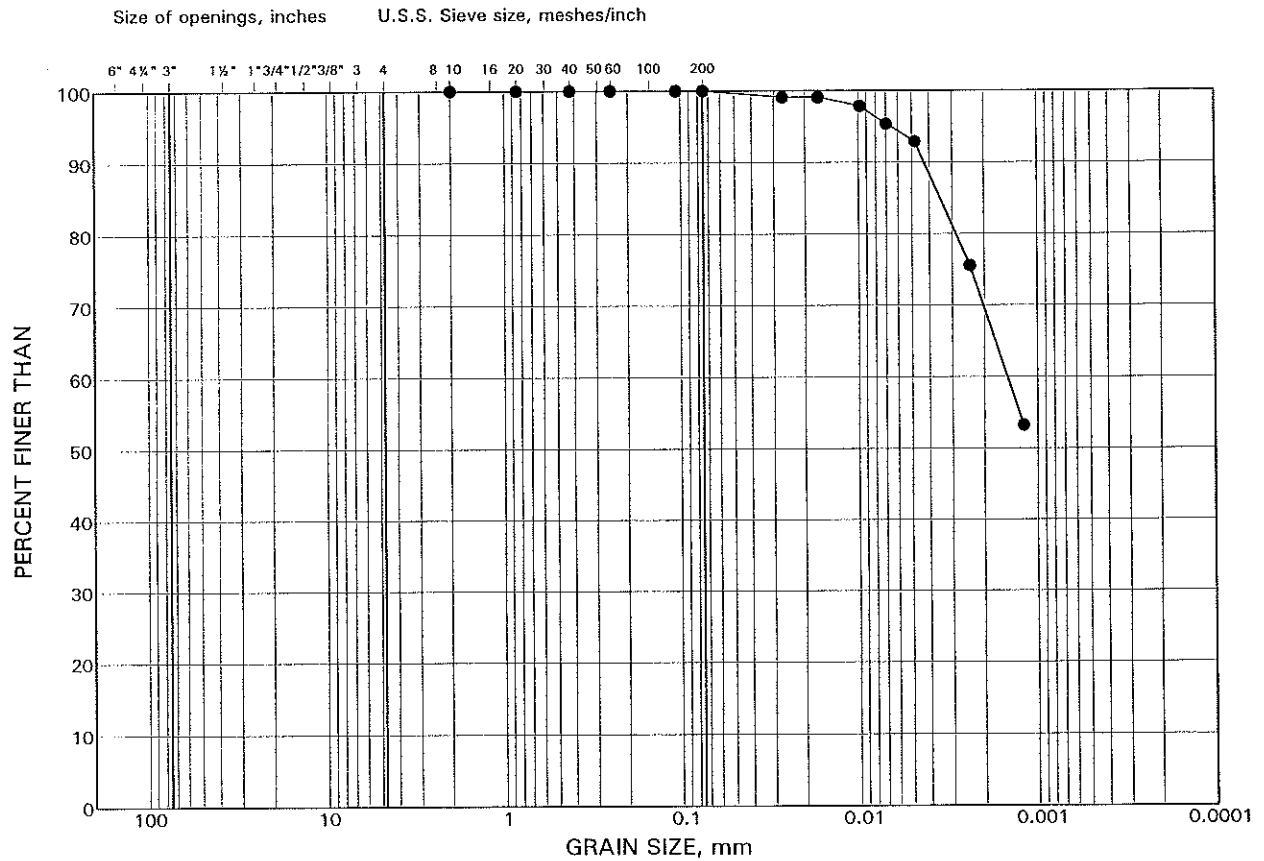
Total Rock Core			
Average	Minimum	Maximum	
137	44	223	MPa
Run #	Average		
1	142.59		
2	194.07		
3	103.29		

Note: Point load test at 46.25 and 48.92 m were performed on Diorite

## Triaxial Test Results

# GRAIN SIZE DISTRIBUTION

FIGURE B29



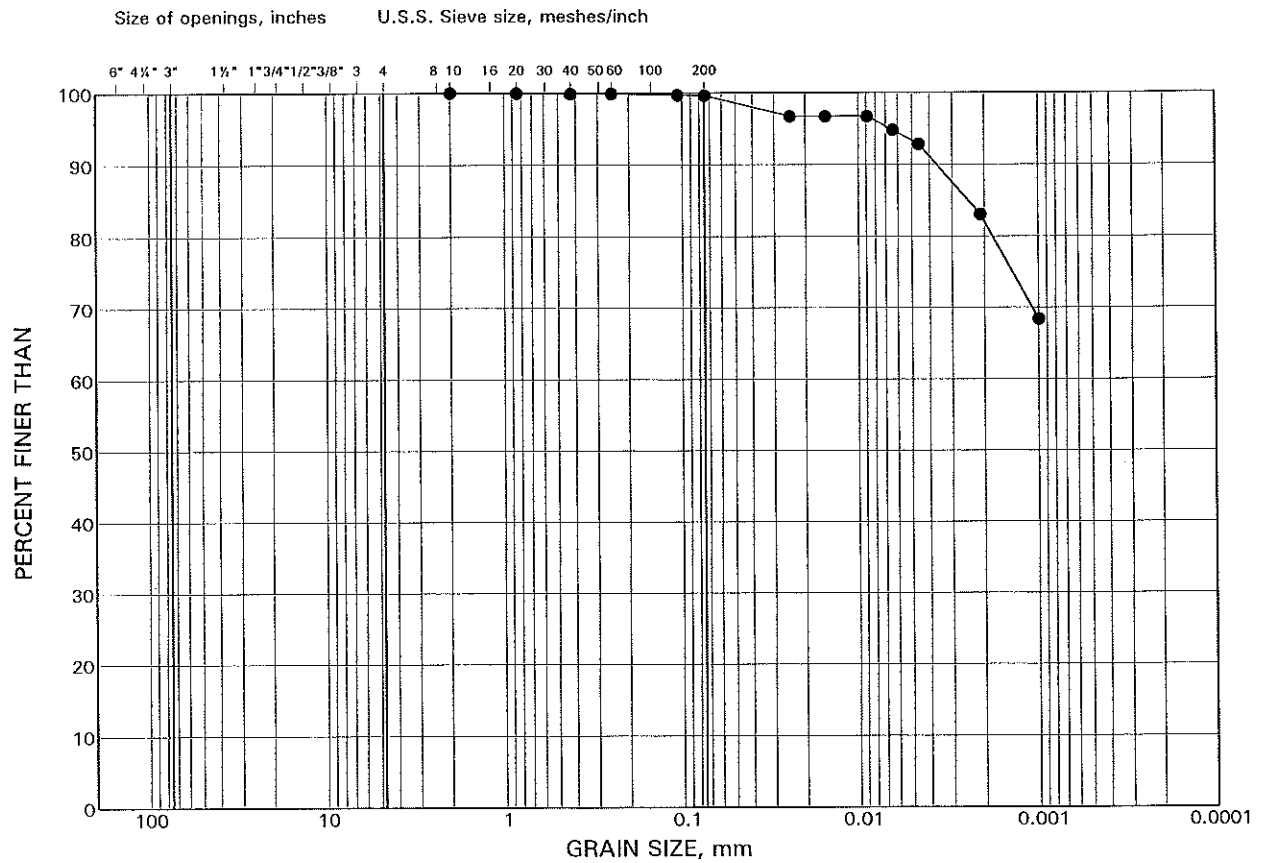
COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	16S	ST #2	36.0-36.6

# GRAIN SIZE DISTRIBUTION

FIGURE B30



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

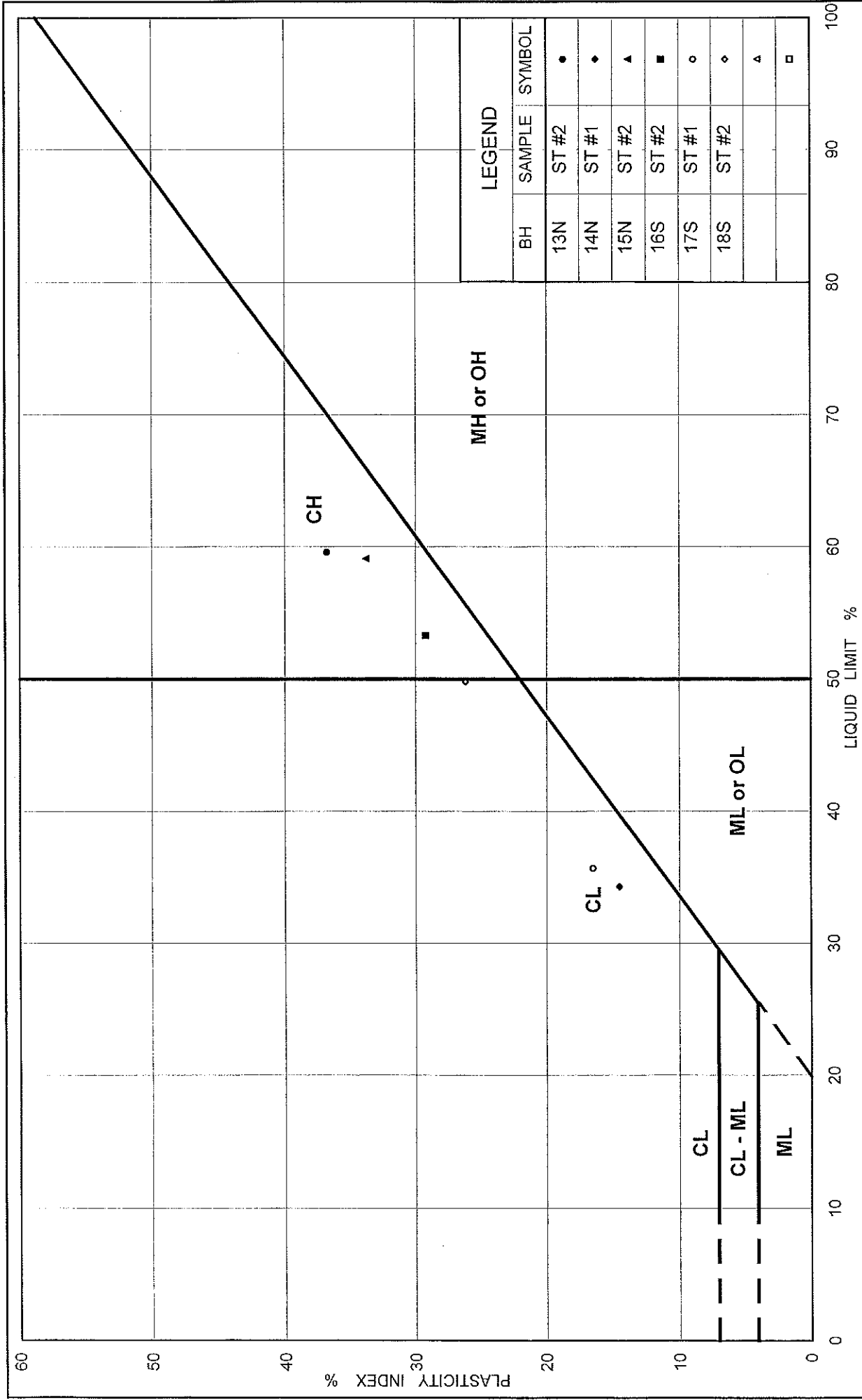
## LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	13N	ST #2	23.8-24.4

TABLE 1

## SUMMARY OF WATER CONTENT DETERMINATIONS

PROJECT NUMBER		04-1116-045			
PROJECT NAME		Thurber / Lab Testing / 15-64-15			
DATE TESTED		June, 2004			
Borehole No.	Sample No.	Depth (ft)	Depth (m)	Water Content (%)	Atterberg Limits LL, PL, PI
13N	ST #2	78.0-80.0	23.77-24.38	46.3%	LL=59.6, PL=22.8, PI=36.8
14N	ST #1	65.0-67.0	19.81-20.42	35.3%	LL=34.3, PL=19.8, PI=14.5
15N	ST #2	110.0-112.0	33.53-34.14	49.4%	LL=59.1, PL=25.3, PI=33.8
16S	ST #2	118.0-120	35.97-36.58	48.8%	LL=53.3, PL=24.1, PI=29.2
17S	ST #1	78.0-80.5	23.77-24.54	31.1%	LL=35.7, PL=19.2, PI=16.5
18S	ST #2	125.0-127.0	38.10-38.71	44.5%	LL=49.8, PL=23.6, PI=26.2





## SPECIFIC GRAVITY TEST RESULTS

### ASTM D 854-00 TEST METHOD A

PROJECT NUMBER	04-1116-045		
PROJECT NAME	Thurber / Lab Testing / 15-64-15		
DATE TESTED	June, 2004		
Borehole	Sample	Specific	
No.	No.	Gravity	
15N	ST #2	2.80	
14N	ST #1	2.78	
17S	ST #1	2.76	
18S	ST #2	2.78	

Note: Test carried out on soil particles <4.75mm using distilled water.

TABLE 2

**CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
SHEET 1 OF 4**

**FIGURE B32**

TEST STAGE	A	B	C
BOREHOLE NUMBER	13N	13N	13N
SAMPLE NUMBER	2	2	2
SPECIMEN DIAMETER, cm	5.02	5.02	5.02
SPECIMEN HEIGHT, cm	10.12	8.04	10.14
WATER CONTENT BEFORE CONSOLIDATION, %	50.9	48.9	57.8
CELL PRESSURE, $\sigma_3$ , kPa	185.0	235.0	355.0
BACK PRESSURE, kPa	135.0	135.0	205.0
PORE PRESSURE PARAMETER "B"	0.97	0.97	0.98
CONSOLIDATION PRESSURE, $\sigma_c$ , kPa	50.0	100.0	150.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	2.4	0.8	3.6
WATER CONTENT AFTER CONSOLIDATION, %	48.9	48.4	54.4
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, DAYS	1	1	1
WATER CONTENT AFTER TEST, %	-	46.7	54.4
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$ , kPa	108.0	208.7	195.8
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ MAXIMUM, %	2.4	4.2	1.9
MAX EFFECTIVE PRINCIPAL STRESS RATIO, $(\sigma_1 / \sigma_3)$ MAXIMUM	3.5	3.3	4.9
DEVIATOR STRESS AT $(\sigma_1 / \sigma_3)$ MAXIMUM, kPa	107.9	208.1	195.4
AXIAL STRAIN AT $(\sigma_1 / \sigma_3)$ MAXIMUM, %	2.4	4.1	1.8
PORE PRESSURE PARAMETER, $A_f$ , AT $(\sigma_1 - \sigma_3)$ MAXIMUM	0.06	0.03	0.51
PORE PRESSURE PARAMETER, $A_f$ , AT $(\sigma_1 / \sigma_3)$ MAXIMUM	0.06	0.04	0.51
NATURAL WATER CONTENT, %	46.3	46.3	52.3
DRY DENSITY, $Mg/m^3$	1.17	1.17	1.12
FILTER DRAINS USED, y/n	y	y	y
TEST NOTES:			
Specimens A and B were multistage			
CHANGED RATE OF STRAIN, %/hr	-	-	-
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %	-	-	-
FAILURE PLANE NUMBER	-	1.0	1.0
ANGLE OF FAILURE, DEGREES	-	55.0	50.0

DATE: 06/15/2004

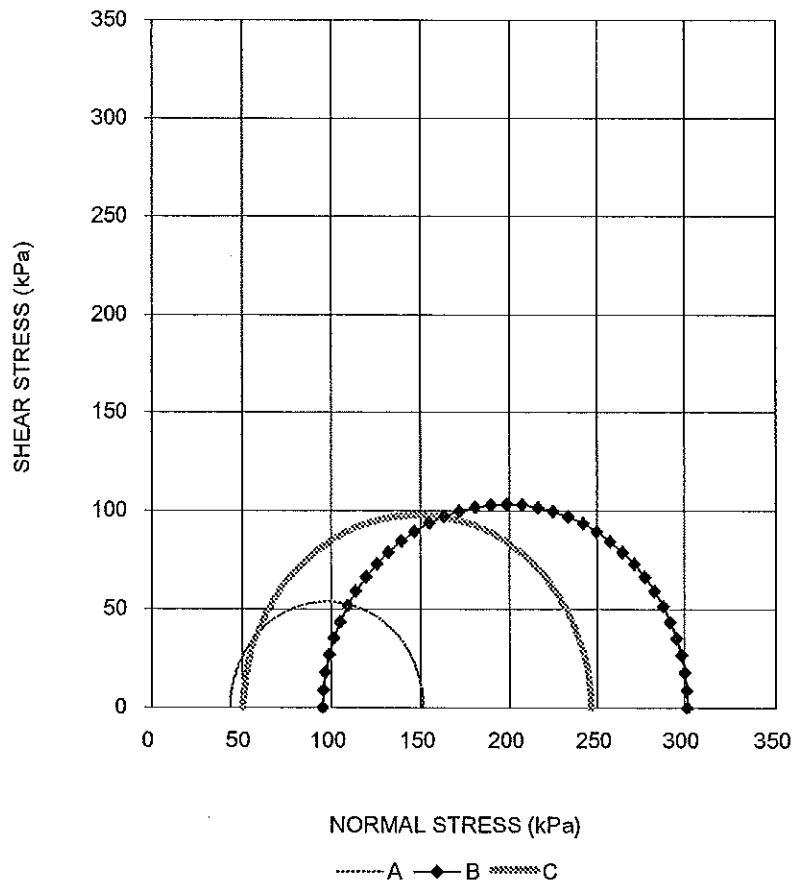
Project No. 04-1116-045

Golder Associates

CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
SHEET 2 OF 4

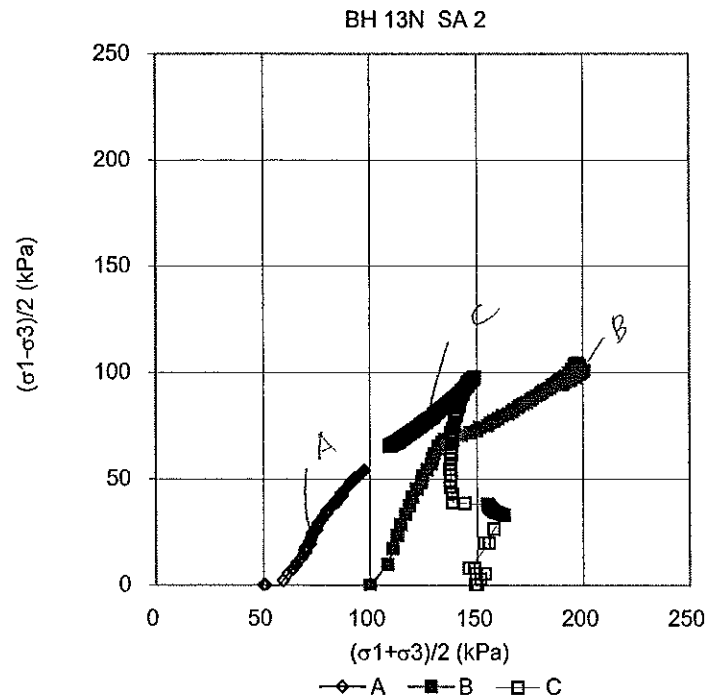
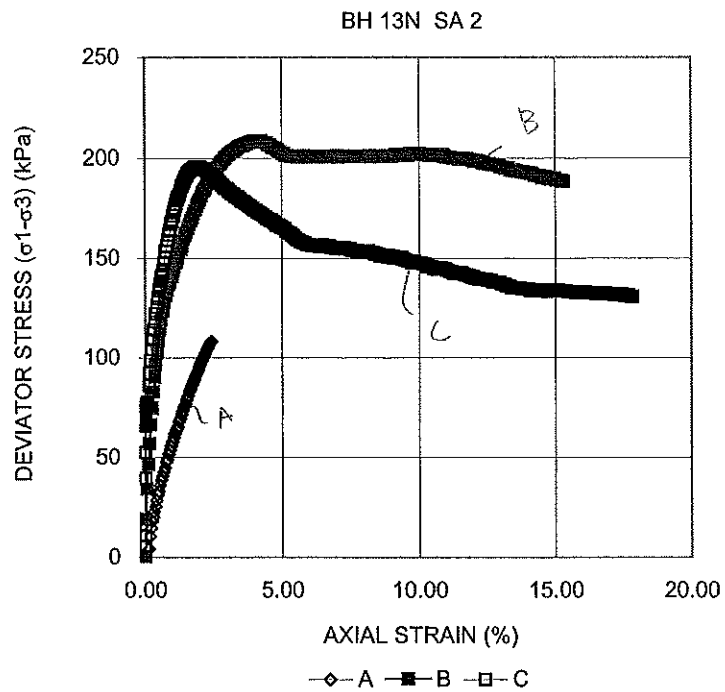
FIGURE B32-1

BH 13N SA 2



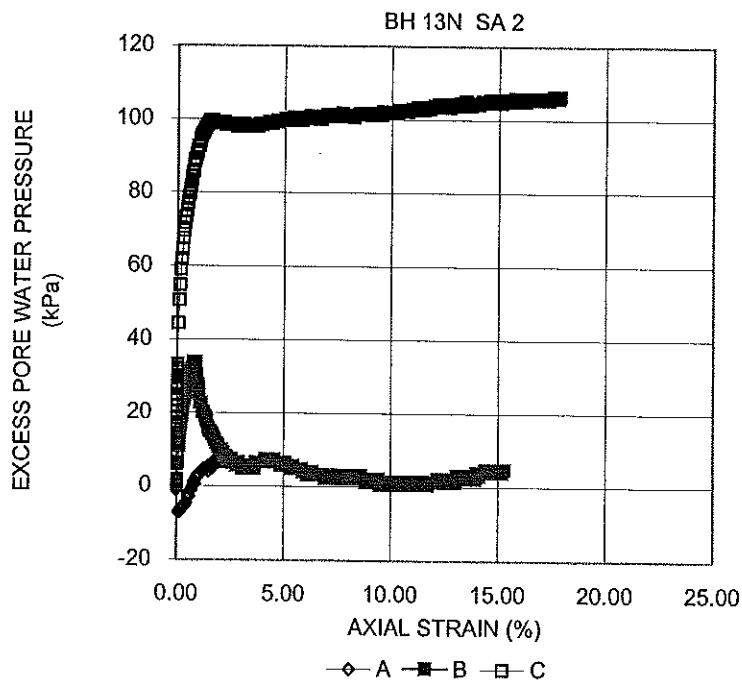
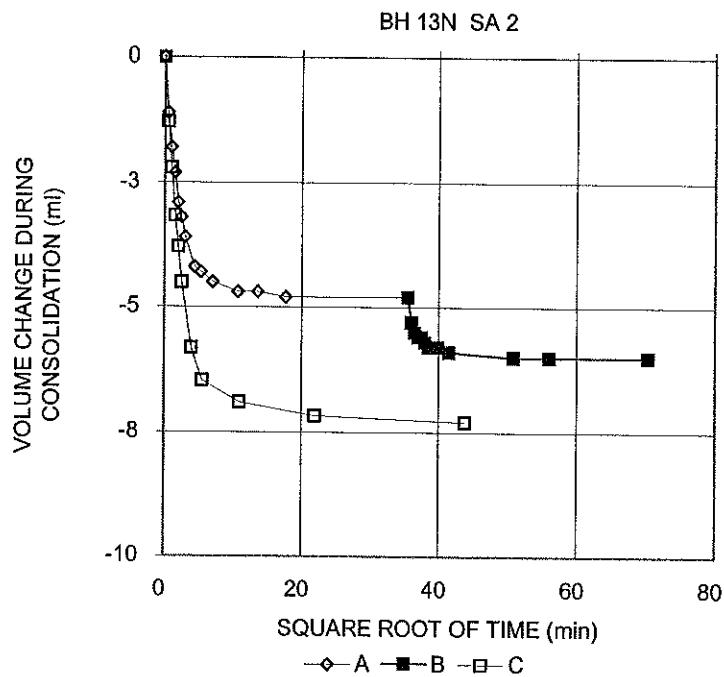
CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
SHEET 3 OF 4

FIGURE B32-2



CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
SHEET 4 OF 4

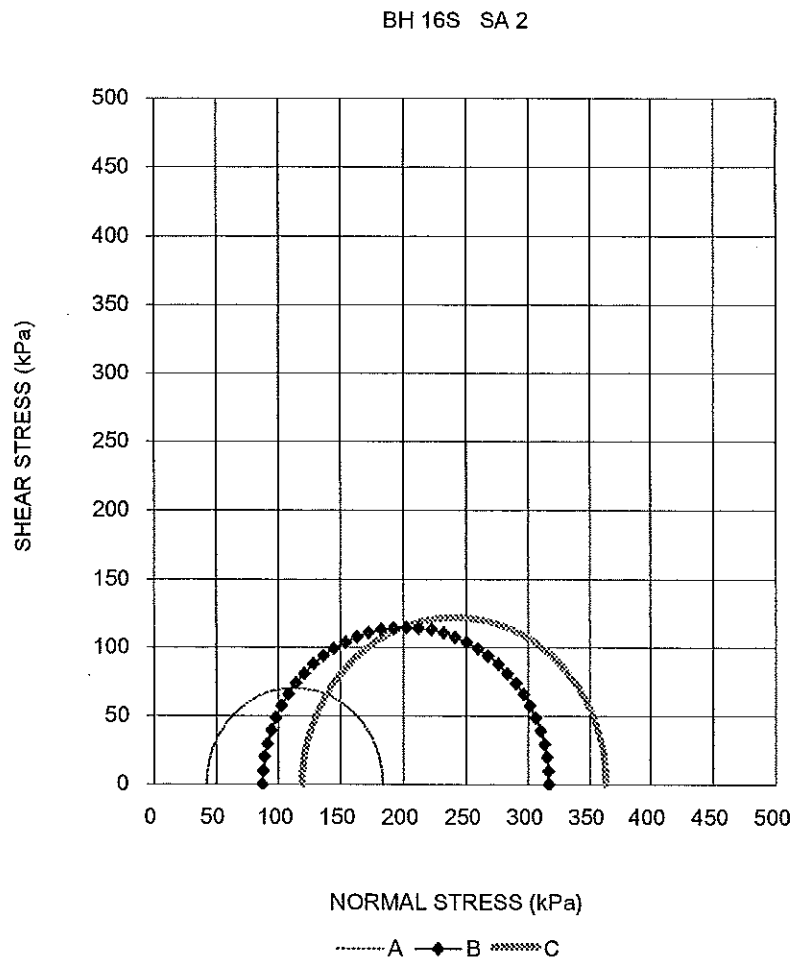
FIGURE B32-3



CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS SHEET 1 OF 4			FIGURE B33
TEST STAGE	A	B	C
BOREHOLE NUMBER	16S	16S	16S
SAMPLE NUMBER	2	2	2
SPECIMEN DIAMETER, cm	5.01	5.00	5.01
SPECIMEN HEIGHT, cm	10.14	10.14	10.13
WATER CONTENT BEFORE CONSOLIDATION, %	54.3	51.4	49.5
CELL PRESSURE, $\sigma_3$ , kPa	305.0	315.0	455.0
BACK PRESSURE, kPa	205.0	135.0	205.0
PORE PRESSURE PARAMETER "B"	0.99	0.97	0.97
CONSOLIDATION PRESSURE, $\sigma_c$ , kPa	100.0	180.0	250.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	3.4	3.2	3.8
WATER CONTENT AFTER CONSOLIDATION, %	51.4	48.7	46.4
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, DAYS	1	1	1
WATER CONTENT AFTER TEST, %	51.4	47.4	45.2
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$ , kPa	141.1	229.7	244.3
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ MAXIMUM, %	2.3	2.2	2.6
MAX EFFECTIVE PRINCIPAL STRESS			
RATIO, $(\sigma_1 / \sigma_3)$ MAXIMUM	4.4	3.8	3.3
DEVIATOR STRESS AT $(\sigma_1 / \sigma_3)$ MAXIMUM, kPa	140.5	223.9	197.0
AXIAL STRAIN AT $(\sigma_1 / \sigma_3)$ MAXIMUM, %	2.1	2.9	7.2
PORE PRESSURE PARAMETER, $A_f$ , AT $(\sigma_1 - \sigma_3)$ MAXIMUM	0.41	0.40	0.54
PORE PRESSURE PARAMETER, $A_f$ , AT $(\sigma_1 / \sigma_3)$ MAXIMUM	0.42	0.44	0.84
NATURAL WATER CONTENT, %	48.8	48.1	45.1
DRY DENSITY, $Mg/m^3$	1.16	1.18	1.21
FILTER DRAINS USED, y/n	y	y	y
TEST NOTES:			
CHANGED RATE OF STRAIN, %/hr	-	-	-
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %	-	-	-
FAILURE PLANE NUMBER	1.0	1.0	-
ANGLE OF FAILURE, DEGREES	55.0	70.0	bulged
DATE: 06/16/2004			
Project No. 04-1116-045	Golder Associates		

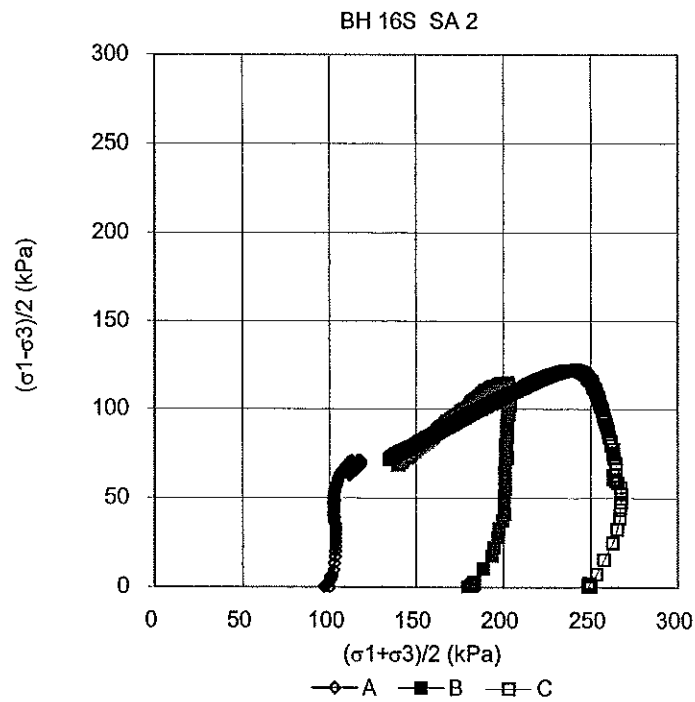
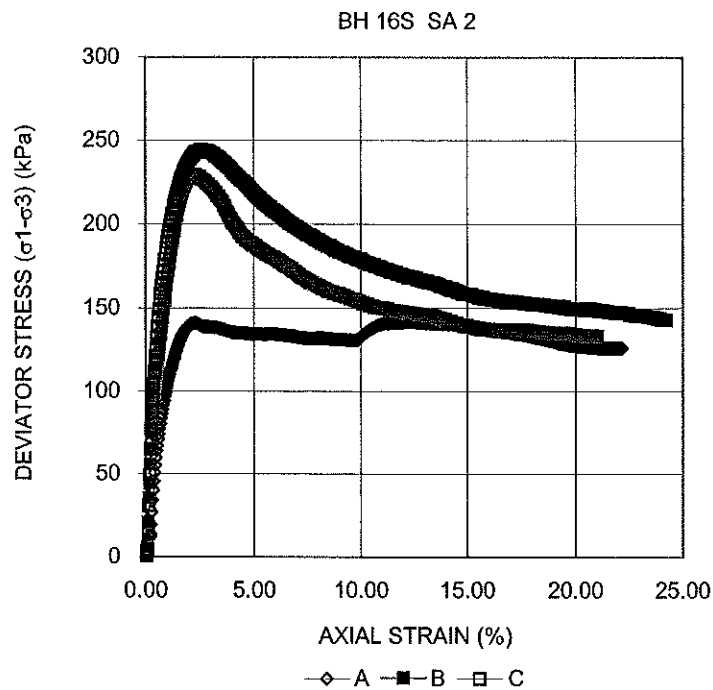
CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
SHEET 2 OF 4

FIGURE B33-1



CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
SHEET 3 OF 4

FIGURE B33-2

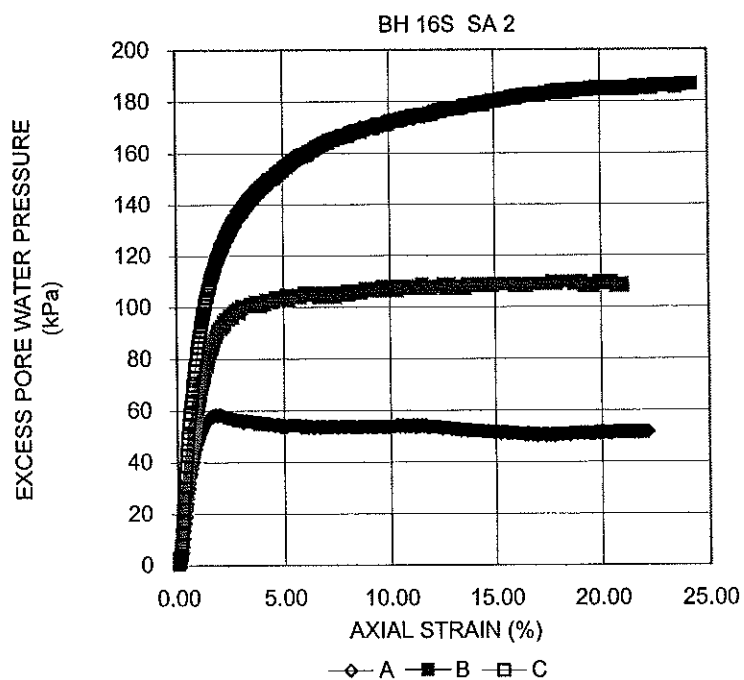
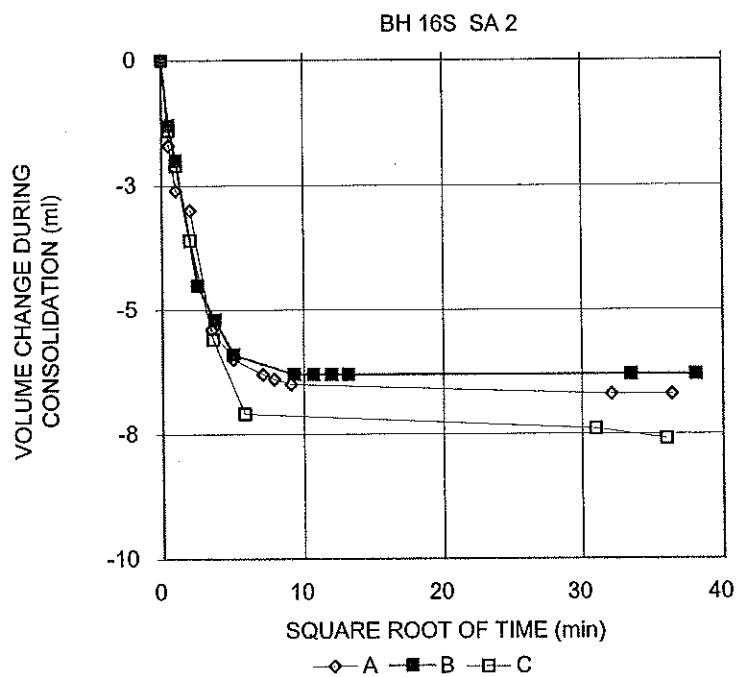




CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS

FIGURE B33-3

SHEET 4 OF 4



Oedometer Test Results

## SPECIFIC GRAVITY TEST RESULTS

### ASTM D 854-00 TEST METHOD A

PROJECT NUMBER	04-1116-045
PROJECT NAME	Thurber / Lab Testing / 15-64-15
DATE TESTED	June, 2004

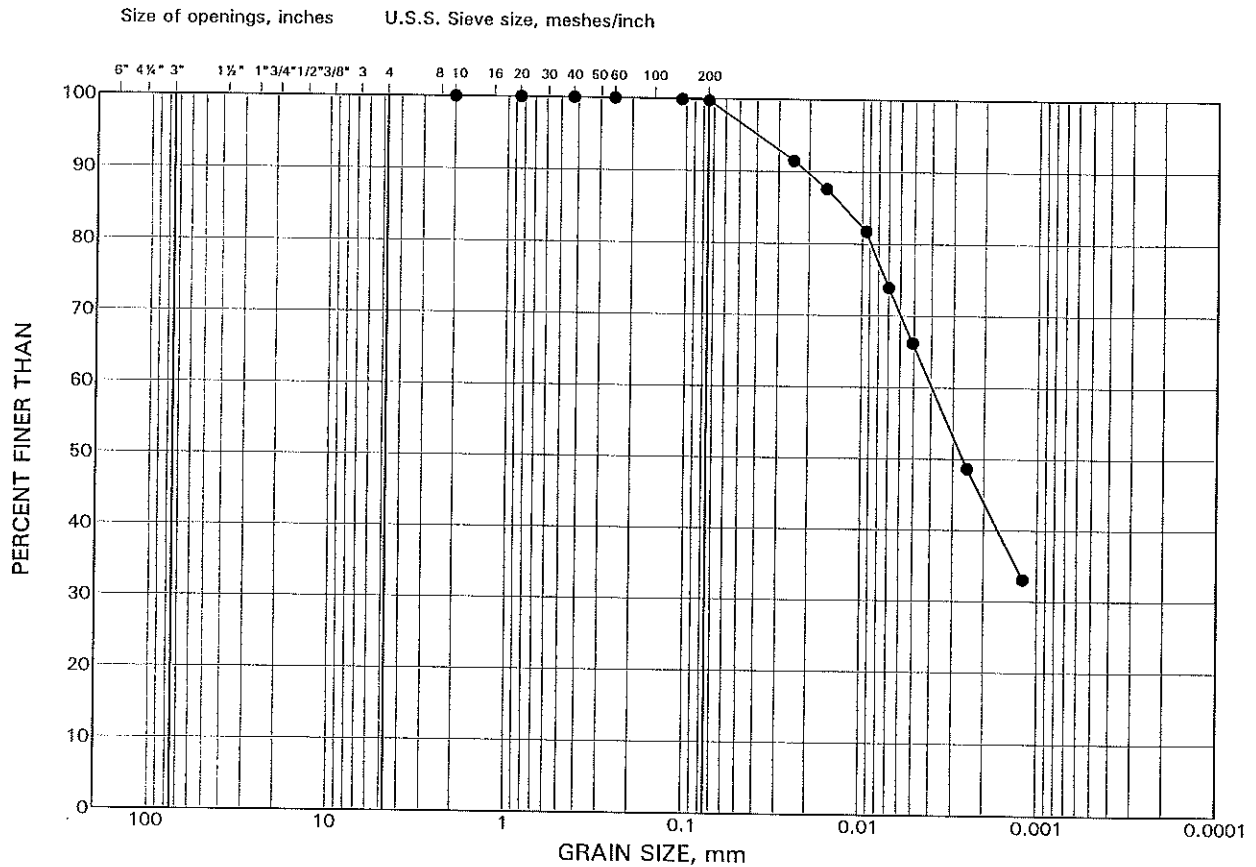
Borehole	Sample	Specific
No.	No.	Gravity
15N	ST #2	2.80
14N	ST #1	2.78
17S	ST #1	2.76
18S	ST #2	2.78

Note: Test carried out on soil particles <4.75mm using distilled water.

TABLE 3

# GRAIN SIZE DISTRIBUTION

FIGURE B34



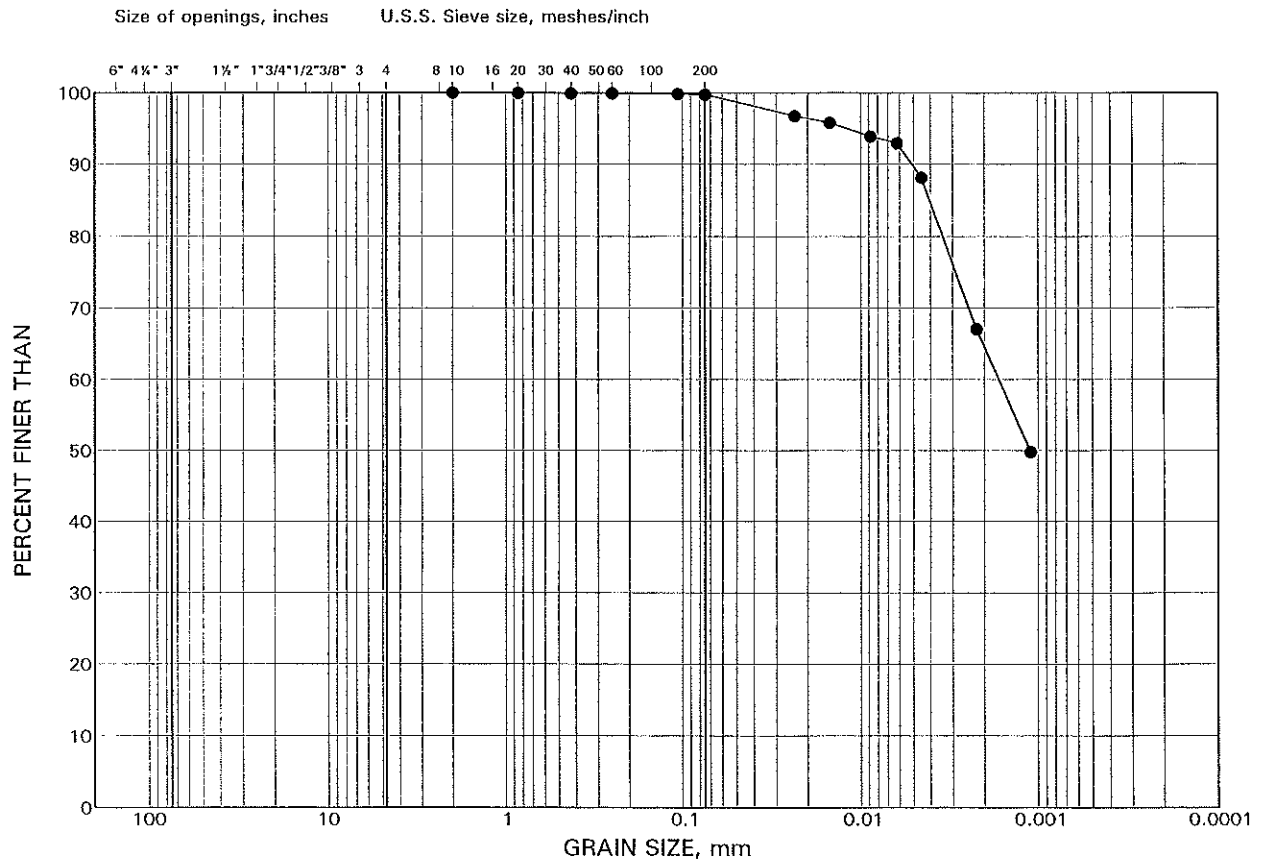
COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	17S	ST #1	23.8-24.5

# GRAIN SIZE DISTRIBUTION

FIGURE B35



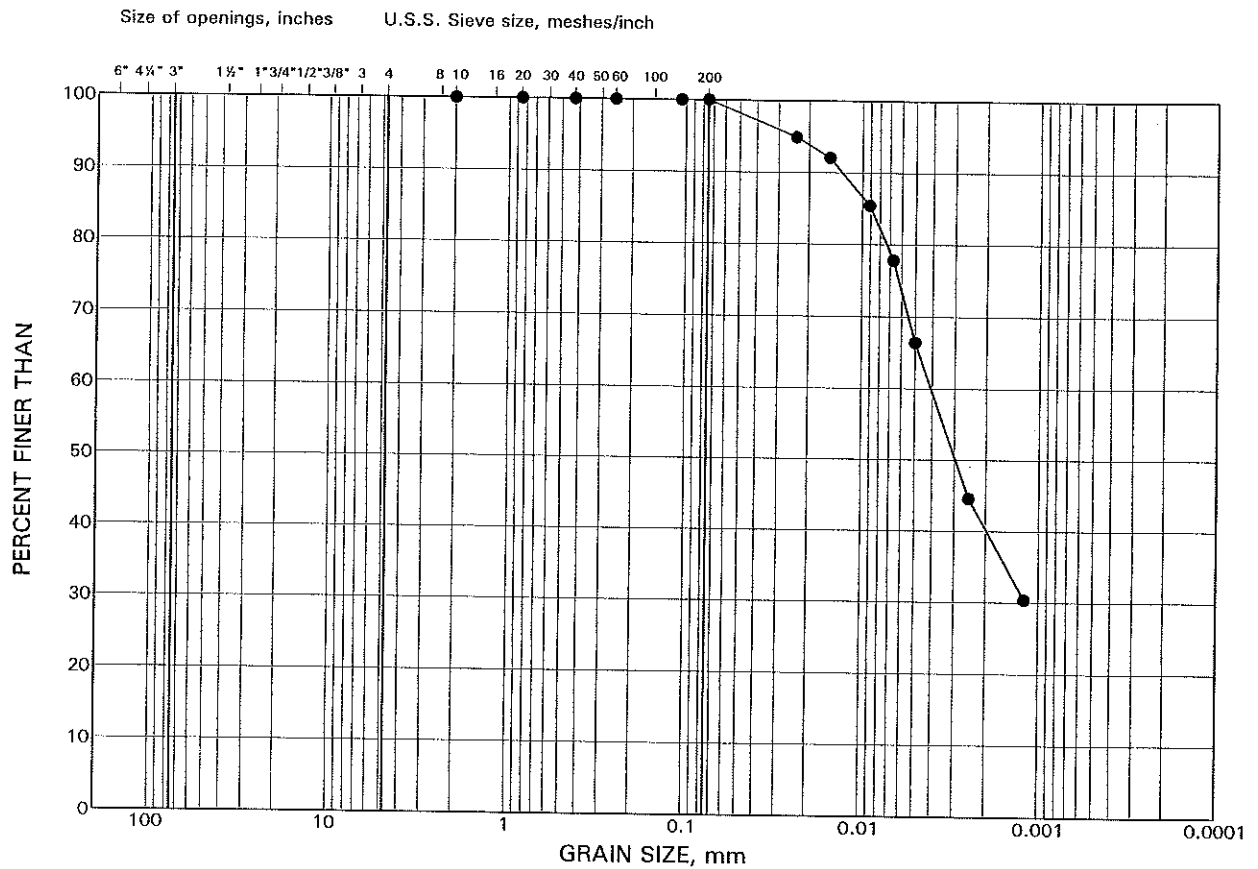
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
	GRAVEL SIZE		SAND SIZE			FINE GRAINED

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	18S	ST #2	38.1-38.7

# GRAIN SIZE DISTRIBUTION

FIGURE B36



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES FINE GRAINED
	GRAVEL SIZE		SAND SIZE			

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	14N	ST #1	19.8-20.4

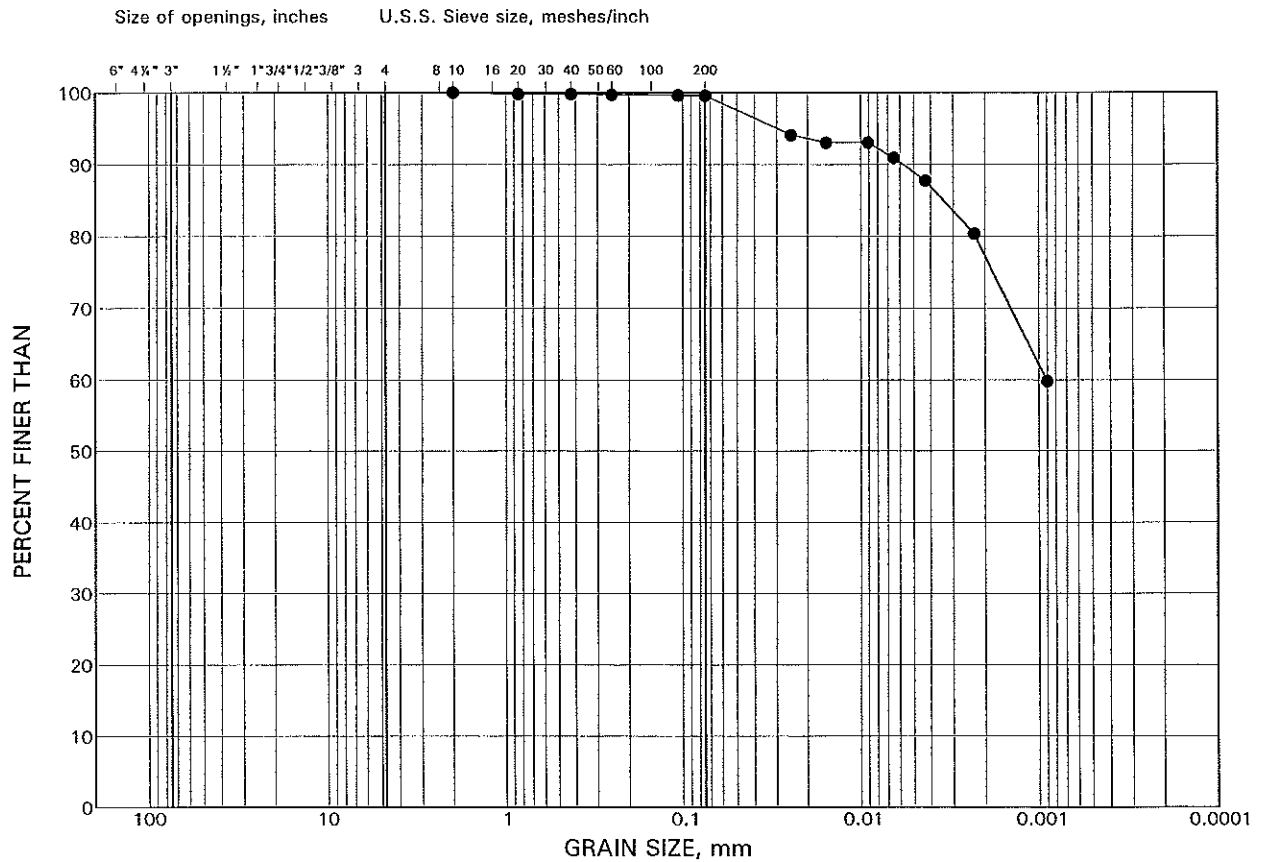
Date 6/16/2004  
Project 04-1116-045

Golder Associates

Prepared by LG  
Checked by *[Signature]*

# GRAIN SIZE DISTRIBUTION

FIGURE B37



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
	GRAVEL SIZE		SAND SIZE			FINE GRAINED

## LEGEND

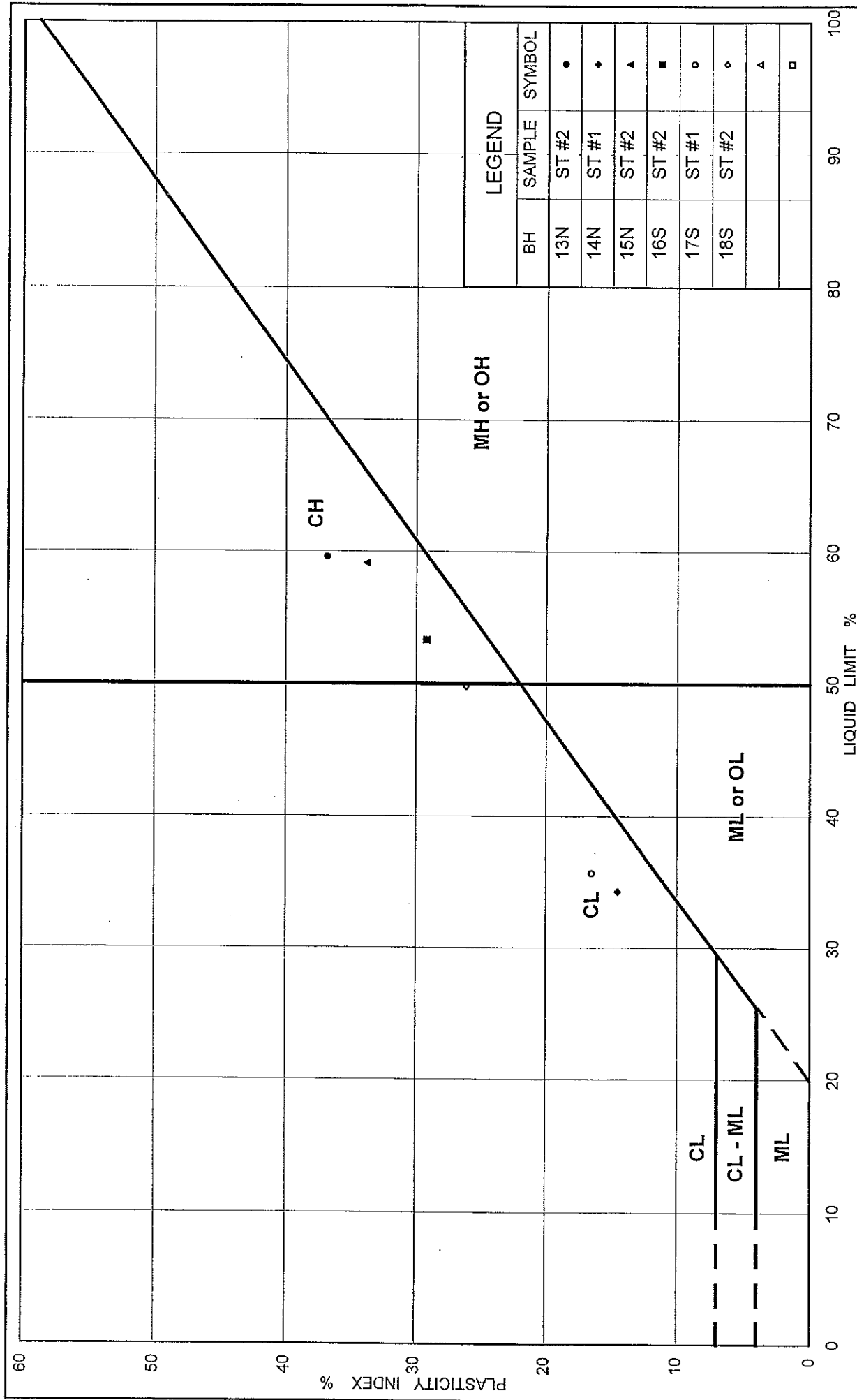
SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	15N	ST #2	33.5-34.1

TABLE 4

## SUMMARY OF WATER CONTENT DETERMINATIONS

PROJECT NUMBER		04-1116-045			
PROJECT NAME		Thurber / Lab Testing / 15-64-15			
DATE TESTED		June, 2004			
Borehole No.	Sample No.	Depth (ft)	Depth (m)	Water Content (%)	Atterberg Limits LL, PL, PI
13N	ST #2	78.0-80.0	23.77-24.38	46.3%	LL=59.6, PL=22.8, PI=36.8
14N	ST #1	65.0-67.0	19.81-20.42	35.3%	LL=34.3, PL=19.8, PI=14.5
15N	ST #2	110.0-112.0	33.53-34.14	49.4%	LL=59.1, PL=25.3, PI=33.8
16S	ST #2	118.0-120	35.97-36.58	48.8%	LL=53.3, PL=24.1, PI=29.2
17S	ST #1	78.0-80.5	23.77-24.54	31.1%	LL=35.7, PL=19.2, PI=16.5
18S	ST #2	125.0-127.0	38.10-38.71	44.5%	LL=49.8, PL=23.6, PI=26.2





# OEDOMETER CONSOLIDATION SUMMARY FIGURE B39

## SAMPLE IDENTIFICATION

Project Number	04-1116-045	Sample Number	ST#1
Borehole Number	14N	Sample Depth, m	19.8-20.4

## TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	06/08/2004		
Date Completed	06/17/2004		

## SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m <sup>3</sup>	18.91
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	14.29
Area, cm <sup>2</sup>	31.65	Specific Gravity, measured	2.78
Volume, cm <sup>3</sup>	60.13	Solids Height, cm	0.996
Water Content, %	32.34	Volume of Solids, cm <sup>3</sup>	31.51
Wet Mass, g	115.94	Volume of Voids, cm <sup>3</sup>	28.62
Dry Mass, g	87.61	Degree of Saturation, %	99.0

## TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
0.00	1.900	0.908	1.900				
4.83	1.901	0.909	1.901				
9.46	1.893	0.901	1.897	158	4.83E-03	9.09E-04	4.30E-07
19.51	1.883	0.891	1.888	108	7.00E-03	5.24E-04	3.59E-07
38.91	1.871	0.879	1.877	98	7.62E-03	3.26E-04	2.43E-07
77.57	1.854	0.862	1.863	72	1.02E-02	2.31E-04	2.32E-07
154.88	1.827	0.835	1.841	40	1.80E-02	1.84E-04	3.23E-07
309.20	1.773	0.781	1.800	60	1.14E-02	1.84E-04	2.07E-07
618.67	1.702	0.709	1.738	103	6.21E-03	1.21E-04	7.35E-08
1237.37	1.636	0.643	1.669	76	7.77E-03	5.61E-05	4.28E-08
2476.14	1.568	0.575	1.602	72	7.56E-03	2.89E-05	2.14E-08
1237.37	1.579	0.586	1.574				
309.20	1.599	0.606	1.589				
77.57	1.618	0.625	1.609				
19.51	1.639	0.646	1.629				
4.83	1.669	0.676	1.654				

Notes:

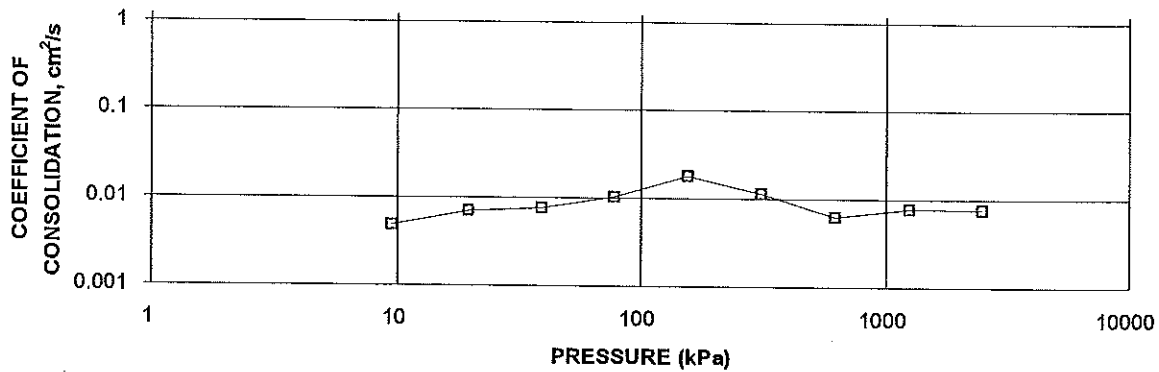
k calculated using cv based on t<sub>90</sub> values.

## SAMPLE DIMENSIONS AND PROPERTIES - FINAL

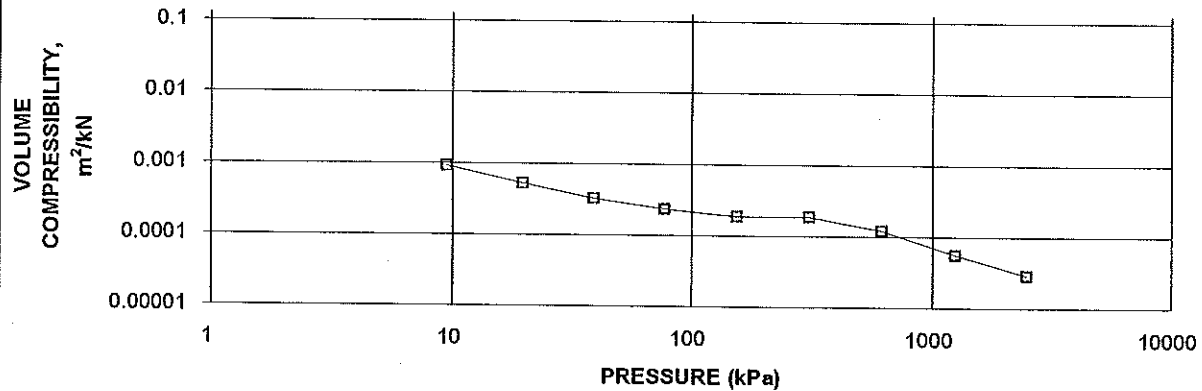
Sample Height, cm	1.67	Unit Weight, kN/m <sup>3</sup>	20.29
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	16.27
Area, cm <sup>2</sup>	31.65	Specific Gravity, measured	2.78
Volume, cm <sup>3</sup>	52.82	Solids Height, cm	0.996
Water Content, %	24.77	Volume of Solids, cm <sup>3</sup>	31.51
Wet Mass, g	109.31	Volume of Voids, cm <sup>3</sup>	21.31
Dry Mass, g	87.61		

# OEDOMETER CONSOLIDATION SUMMARY FIGURE B40

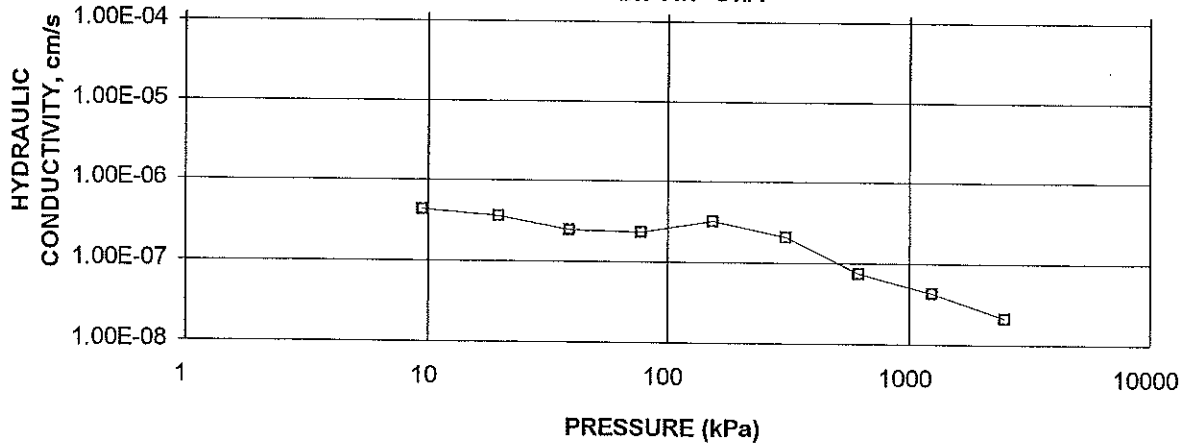
CONSOLIDATION TEST  
CV  $\text{cm}^2/\text{s}$  VS PRESSURE (kPa)  
BH 14N ST#1



CONSOLIDATION TEST  
MV  $\text{m}^2/\text{kN}$  vs PRESSURE (kPa)  
BH 14N ST#1



CONSOLIDATION TEST  
HYDRAULIC CONDUCTIVITY vs PRESSURE  
BH 14N ST#1

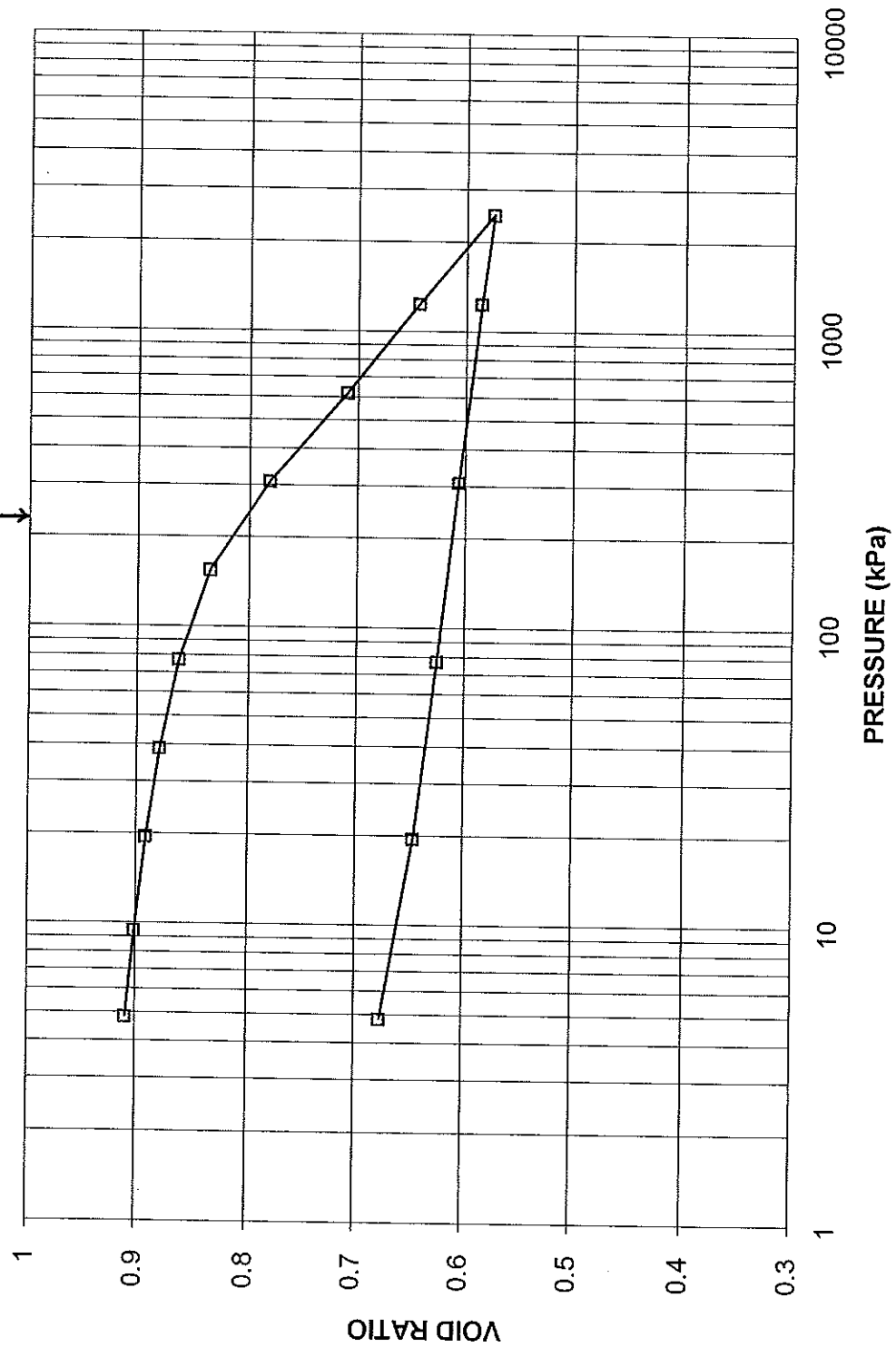


# CONSOLIDATION TEST VOID RATIO VS. LOG PRESSURE

FIGURE B41

CONSOLIDATION TEST  
VOID RATIO vs PRESSURE  
BH 14N ST#1

$p_o \equiv p'_c$



# OEDOMETER CONSOLIDATION SUMMARY FIGURE B42

## SAMPLE IDENTIFICATION

Project Number	04-1116-045	Sample Number	ST#2
Borehole Number	15N	Sample Depth, m	33.5-34.1

## TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	9		
Date Started	06/02/2004		
Date Completed	06/13/2004		

## SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.92	Unit Weight, kN/m <sup>3</sup>	16.98
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m <sup>3</sup>	11.29
Area, cm <sup>2</sup>	31.52	Specific Gravity, measured	2.80
Volume, cm <sup>3</sup>	60.36	Solids Height, cm	0.787
Water Content, %	50.37	Volume of Solids, cm <sup>3</sup>	24.82
Wet Mass, g	104.49	Volume of Voids, cm <sup>3</sup>	35.54
Dry Mass, g	69.49	Degree of Saturation, %	98.5

## TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
0.00	1.915	1.432	1.915				
4.87	1.913	1.429	1.914	3	2.59E-01	2.57E-04	6.53E-06
9.55	1.910	1.426	1.911	44	1.76E-02	2.90E-04	5.00E-07
19.50	1.905	1.419	1.908	40	1.93E-02	2.62E-04	4.96E-07
38.82	1.895	1.407	1.900	129	5.93E-03	2.70E-04	1.57E-07
77.80	1.876	1.383	1.886	76	9.92E-03	2.55E-04	2.47E-07
155.39	1.846	1.345	1.861	197	3.73E-03	2.02E-04	7.37E-08
310.90	1.793	1.277	1.820	240	2.92E-03	1.78E-04	5.10E-08
621.39	1.654	1.101	1.724	960	6.56E-04	2.34E-04	1.50E-08
1243.77	1.500	0.905	1.577	1500	3.51E-04	1.29E-04	4.45E-09
2485.17	1.383	0.756	1.442	671	6.57E-04	4.92E-05	3.17E-09
1243.77	1.397	0.774	1.390				
310.90	1.441	0.830	1.419				
77.80	1.499	0.904	1.470				
19.50	1.560	0.981	1.530				
4.87	1.588	1.017	1.574				

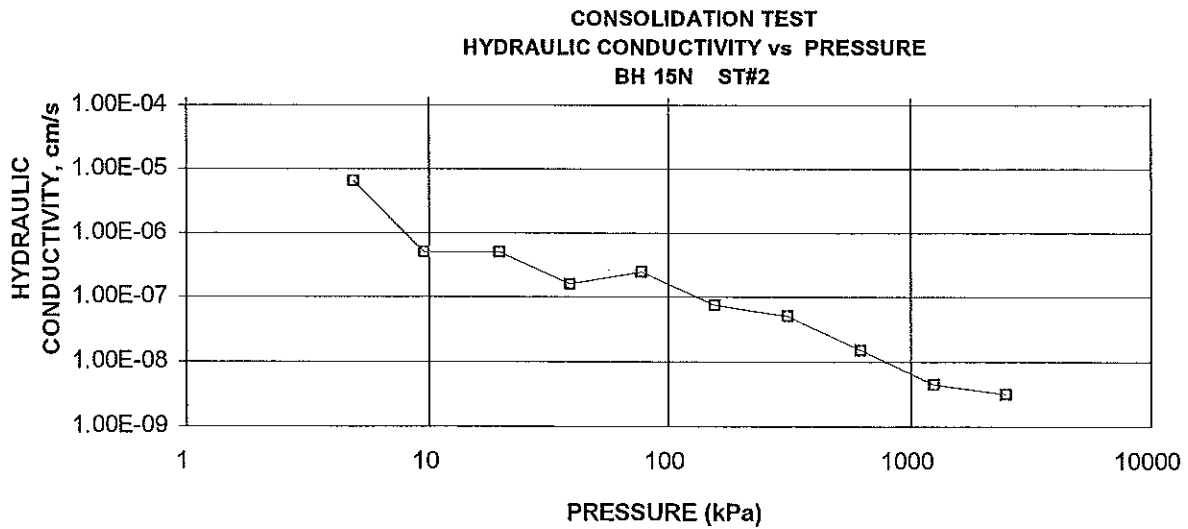
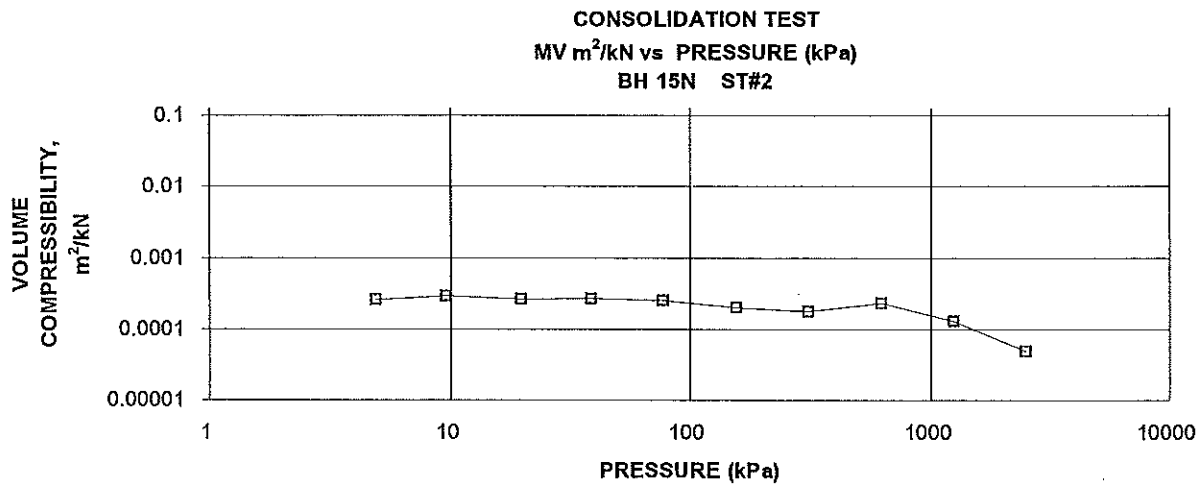
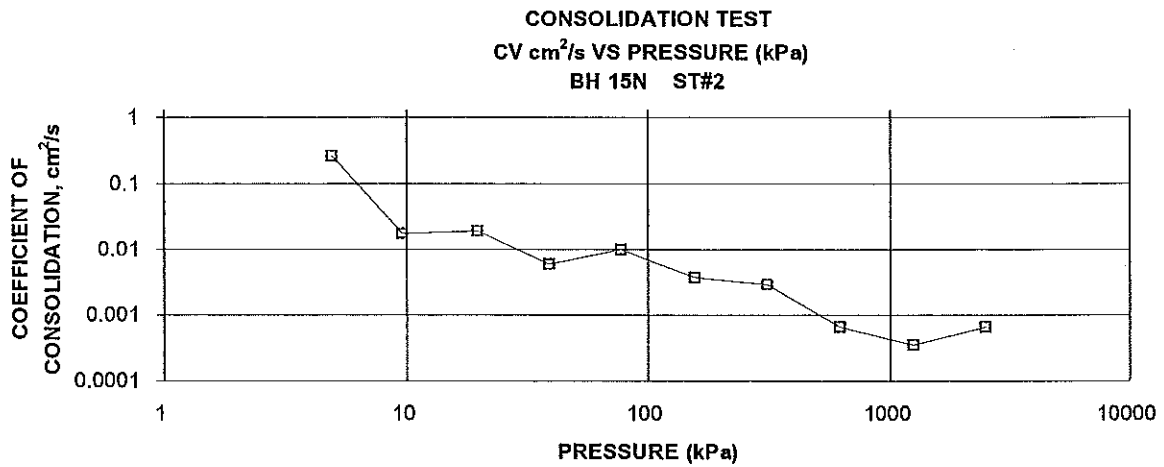
Notes:

k calculated using cv based on  $g_b$  values.

## SAMPLE DIMENSIONS AND PROPERTIES - FINAL

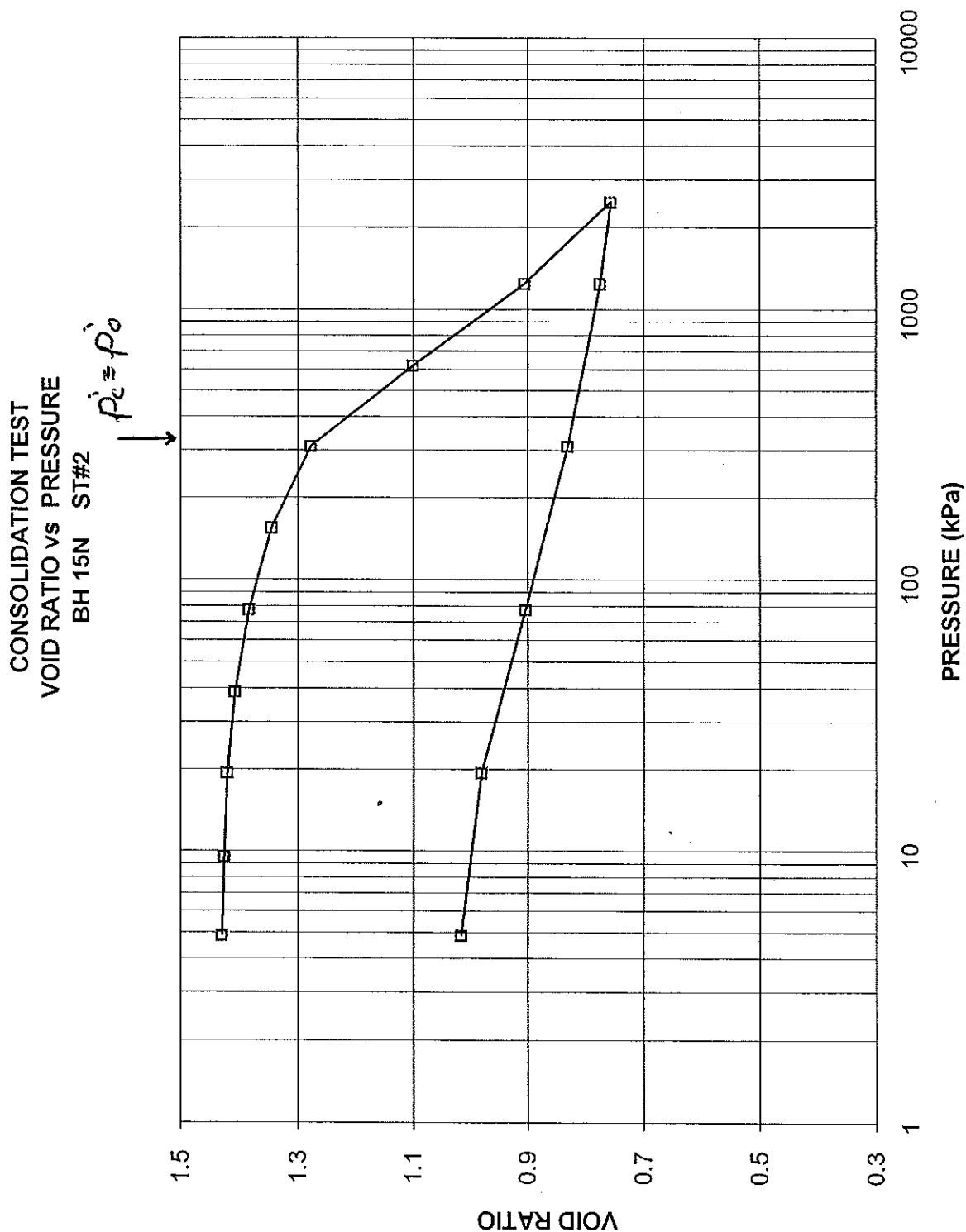
Sample Height, cm	1.59	Unit Weight, kN/m <sup>3</sup>	18.65
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m <sup>3</sup>	13.61
Area, cm <sup>2</sup>	31.52	Specific Gravity, measured	2.80
Volume, cm <sup>3</sup>	50.05	Solids Height, cm	0.787
Water Content, %	37.00	Volume of Solids, cm <sup>3</sup>	24.82
Wet Mass, g	95.20	Volume of Voids, cm <sup>3</sup>	25.24
Dry Mass, g	69.49		

# OEDOMETER CONSOLIDATION SUMMARY FIGURE B43



# CONSOLIDATION TEST VOID RATIO VS. LOG PRESSURE

FIGURE B44



BOREHOLE 15N SAMPLE NUMBER ST #2

APPLIED PRESSURE = 310.9 kPa

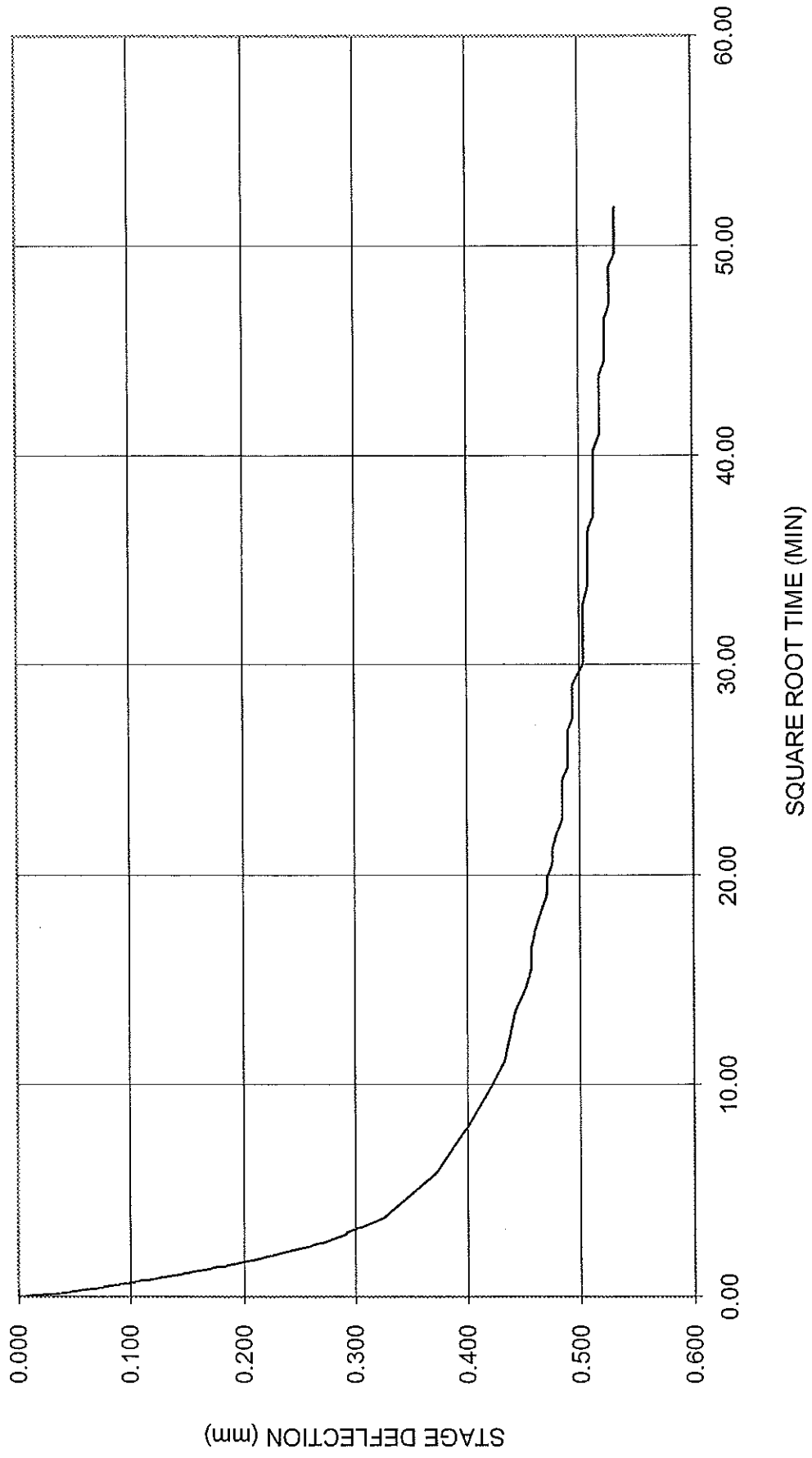
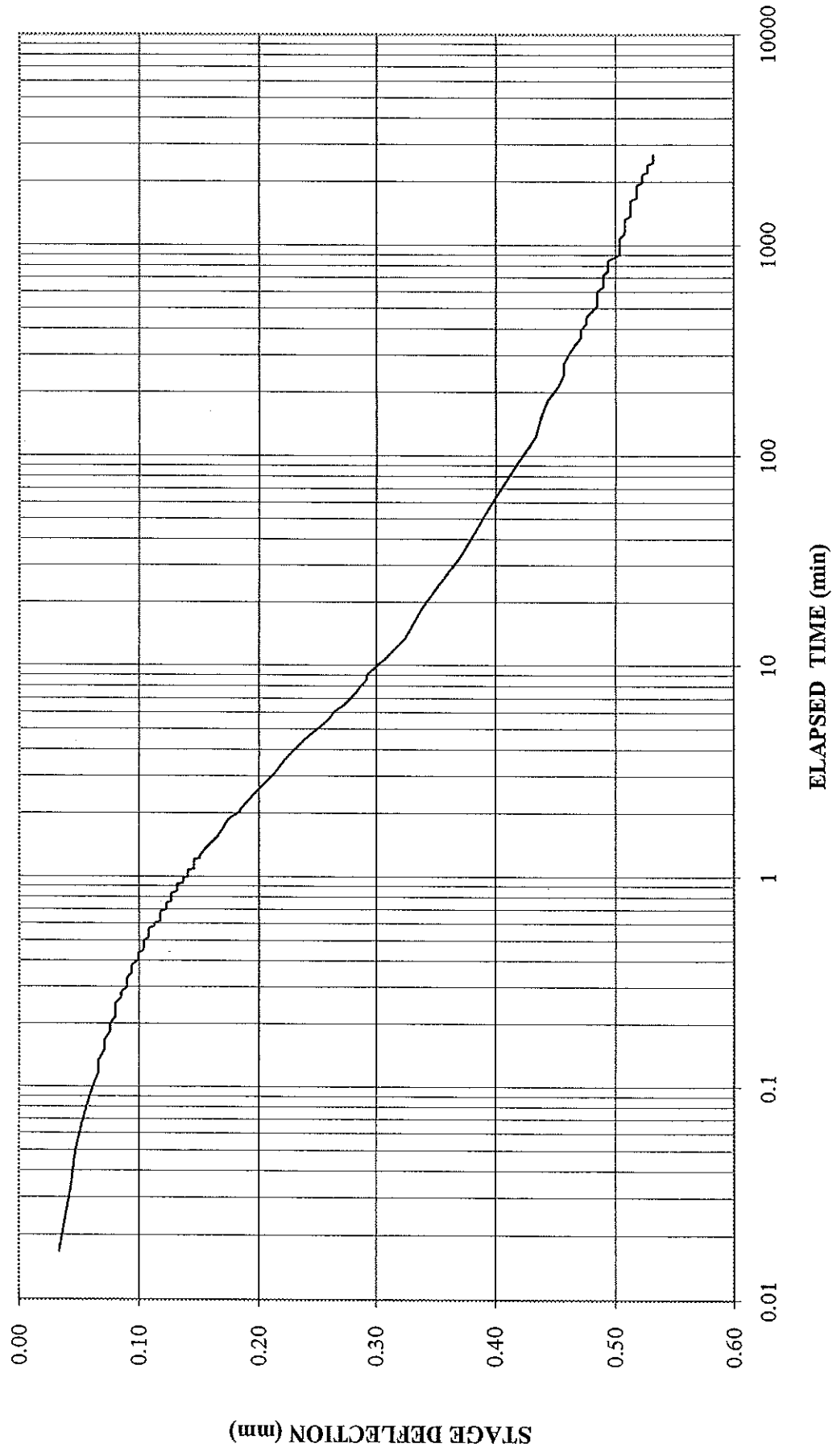


FIGURE B45



**BOREHOLE 15N SAMPLE NUMBER ST #2**

**APPLIED PRESSURE = 310.9 kPa**



**FIGURE B46**

# OEDOMETER CONSOLIDATION SUMMARY FIGURE B47

## SAMPLE IDENTIFICATION

Project Number	04-1116-045	Sample Number	ST#1
Borehole Number	17S	Sample Depth, m	23.8-24.5

## TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	6		
Date Started	06/08/2004		
Date Completed	06/19/2004		

## SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m <sup>3</sup>	18.62
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	13.89
Area, cm <sup>2</sup>	31.67	Specific Gravity, measured	2.76
Volume, cm <sup>3</sup>	60.17	Solids Height, cm	0.975
Water Content, %	34.06	Volume of Solids, cm <sup>3</sup>	30.87
Wet Mass, g	114.23	Volume of Voids, cm <sup>3</sup>	29.30
Dry Mass, g	85.21	Degree of Saturation, %	99.1

## TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
0.00	1.900	0.949	1.900				
4.75	1.898	0.947	1.899	38	2.01E-02	2.22E-04	4.37E-07
9.54	1.895	0.944	1.897	108	7.06E-03	3.30E-04	2.28E-07
19.25	1.889	0.938	1.892	135	5.62E-03	3.25E-04	1.79E-07
38.68	1.879	0.927	1.884	85	8.85E-03	2.71E-04	2.35E-07
77.38	1.865	0.913	1.872	49	1.52E-02	1.90E-04	2.83E-07
154.68	1.845	0.893	1.855	68	1.07E-02	1.36E-04	1.43E-07
309.13	1.808	0.855	1.827	119	5.94E-03	1.26E-04	7.34E-08
618.25	1.726	0.771	1.767	304	2.18E-03	1.40E-04	2.98E-08
1237.60	1.632	0.674	1.679	240	2.49E-03	7.99E-05	1.95E-08
2475.59	1.549	0.589	1.591	141	3.80E-03	3.53E-05	1.32E-08
1237.60	1.560	0.600	1.555				
309.13	1.578	0.619	1.569				
77.38	1.601	0.642	1.590				
19.25	1.629	0.671	1.615				
4.75	1.661	0.704	1.645				

**Notes:**

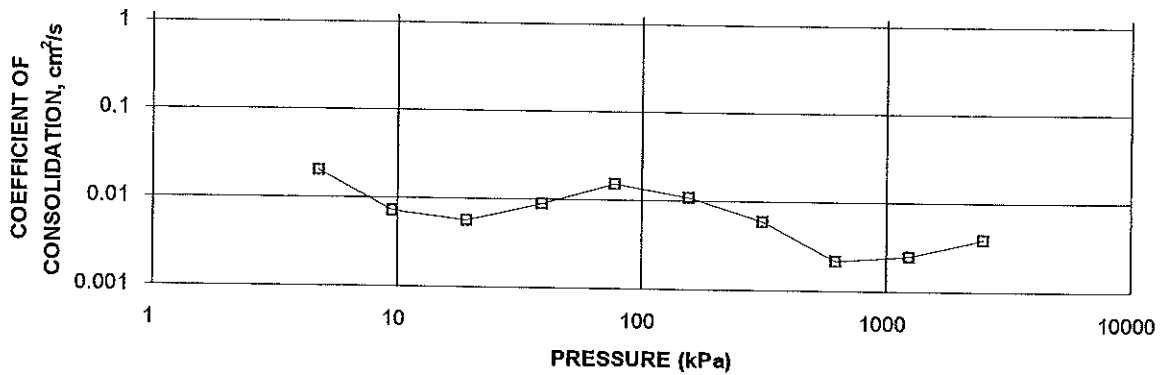
k calculated using cv based on t<sub>90</sub> values.

## SAMPLE DIMENSIONS AND PROPERTIES - FINAL

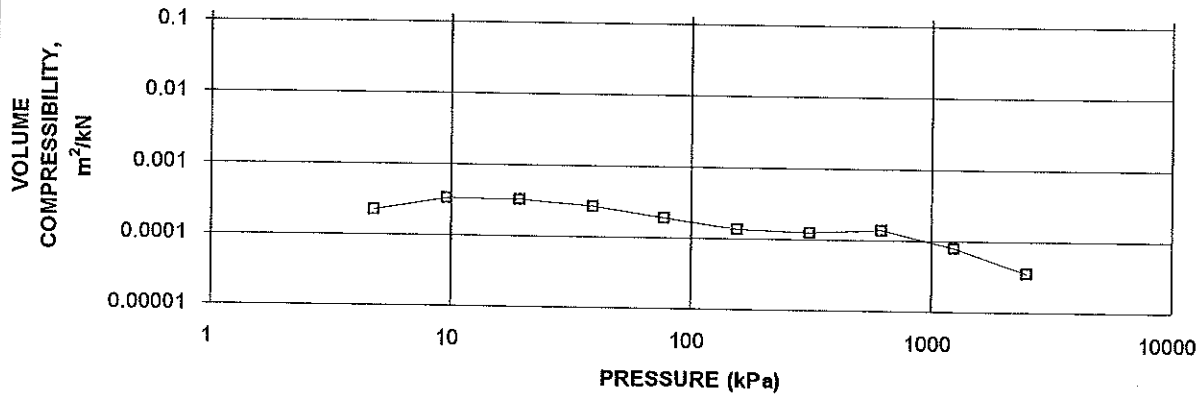
Sample Height, cm	1.66	Unit Weight, kN/m <sup>3</sup>	20.05
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	15.89
Area, cm <sup>2</sup>	31.67	Specific Gravity, measured	2.76
Volume, cm <sup>3</sup>	52.60	Solids Height, cm	0.975
Water Content, %	26.21	Volume of Solids, cm <sup>3</sup>	30.87
Wet Mass, g	107.54	Volume of Voids, cm <sup>3</sup>	21.73
Dry Mass, g	85.21		

# OEDOMETER CONSOLIDATION SUMMARY FIGURE B48

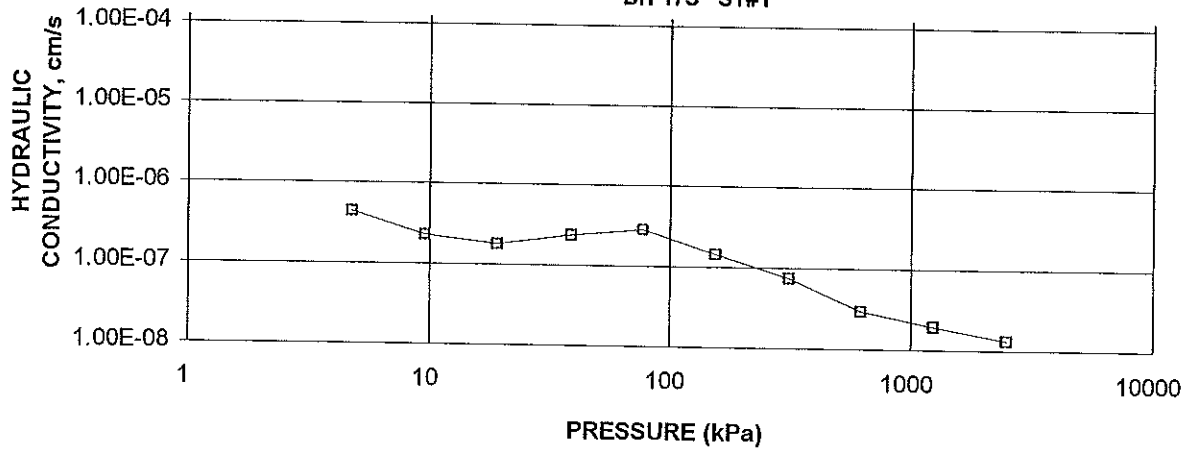
CONSOLIDATION TEST  
CV  $\text{cm}^2/\text{s}$  VS PRESSURE (kPa)  
BH 17S ST#1



CONSOLIDATION TEST  
MV  $\text{m}^2/\text{kN}$  vs PRESSURE (kPa)  
BH 17S ST#1

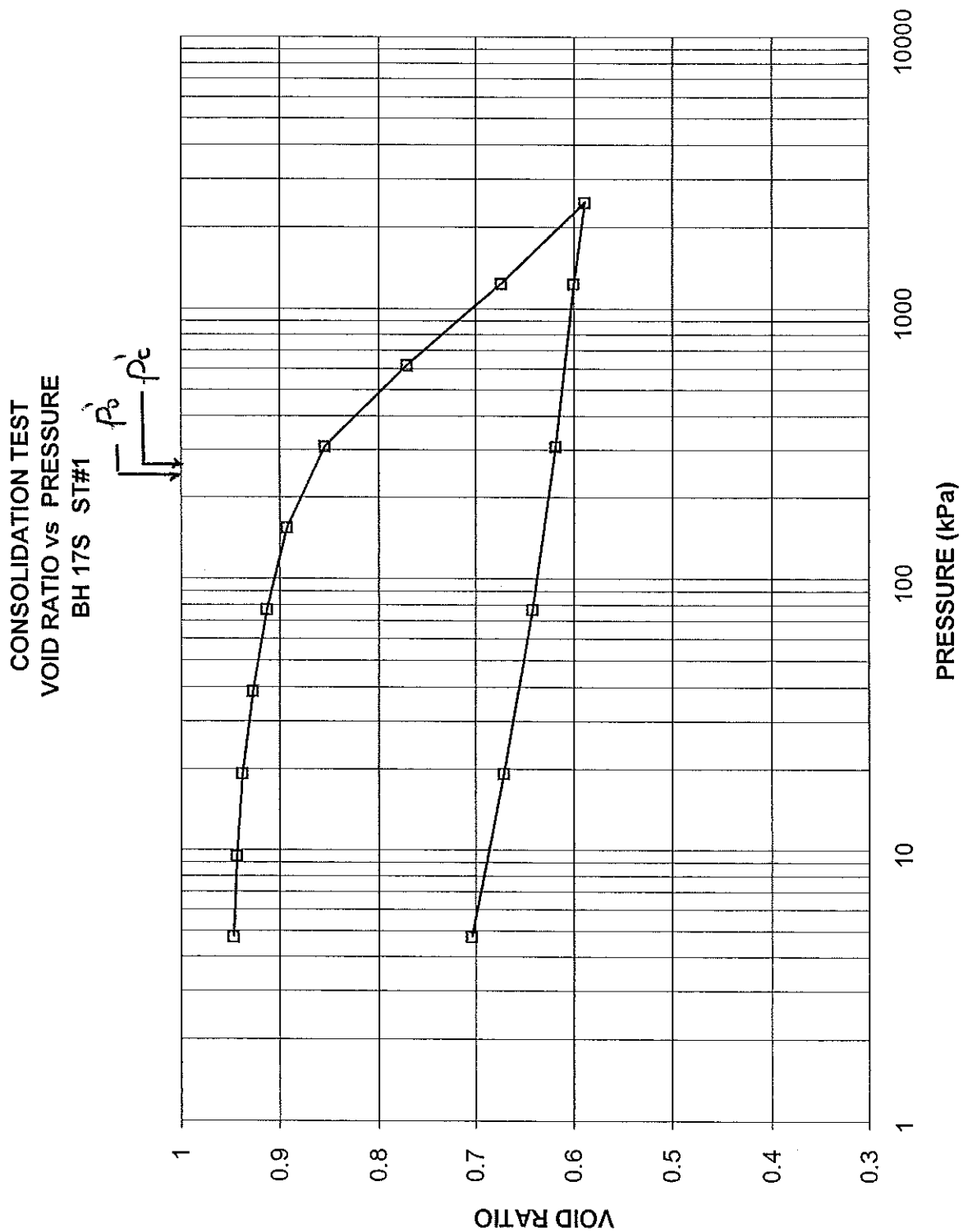


CONSOLIDATION TEST  
HYDRAULIC CONDUCTIVITY vs PRESSURE  
BH 17S ST#1



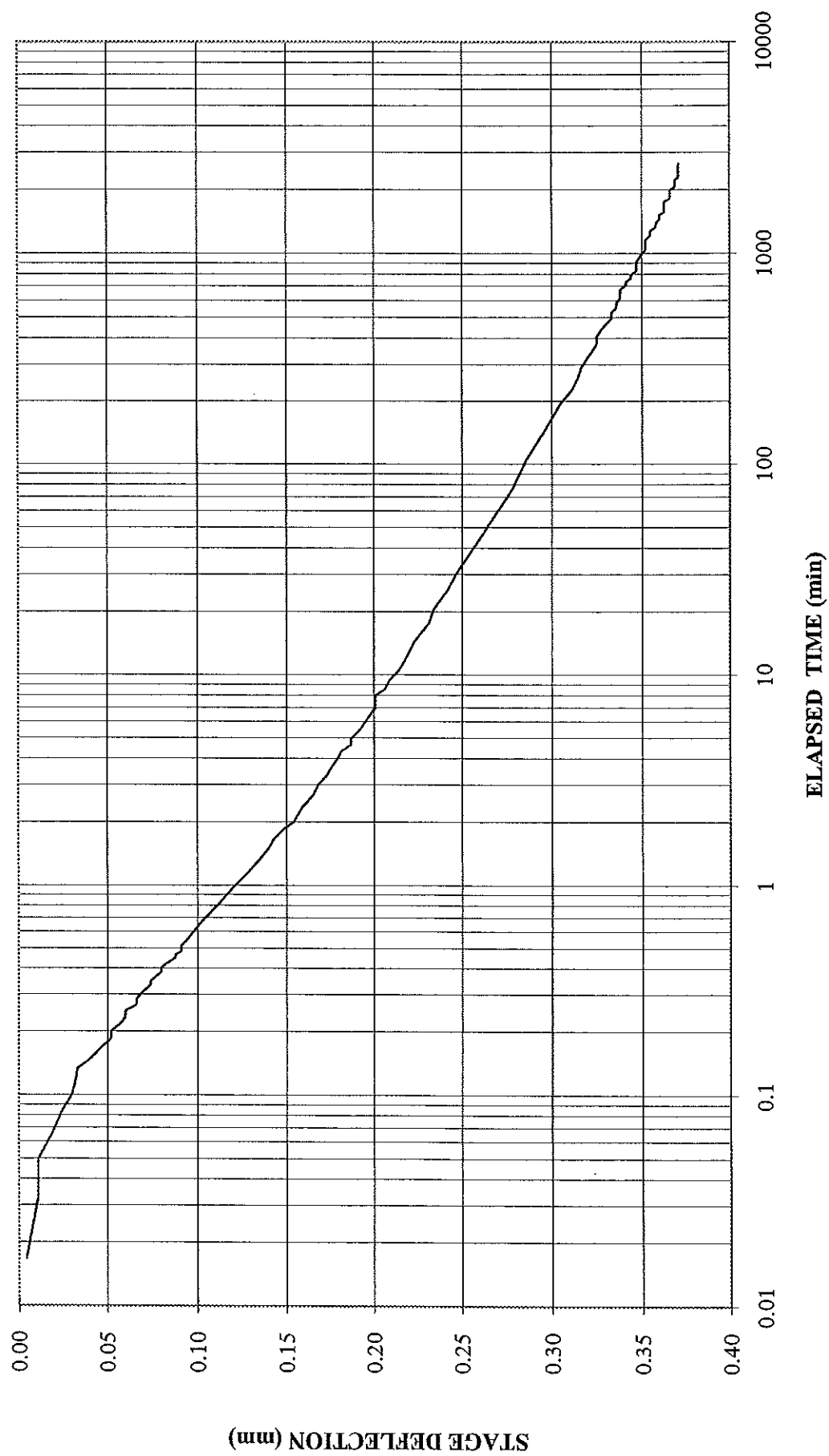
CONSOLIDATION TEST  
VOID RATIO VS. LOG PRESSURE

FIGURE B49



**BOREHOLE 17S SAMPLE NUMBER ST #1**

**APPLIED PRESSURE = 309.1 kPa**



**FIGURE B50**

BOREHOLE 17S SAMPLE NUMBER ST #1

APPLIED PRESSURE = 309.1 kPa

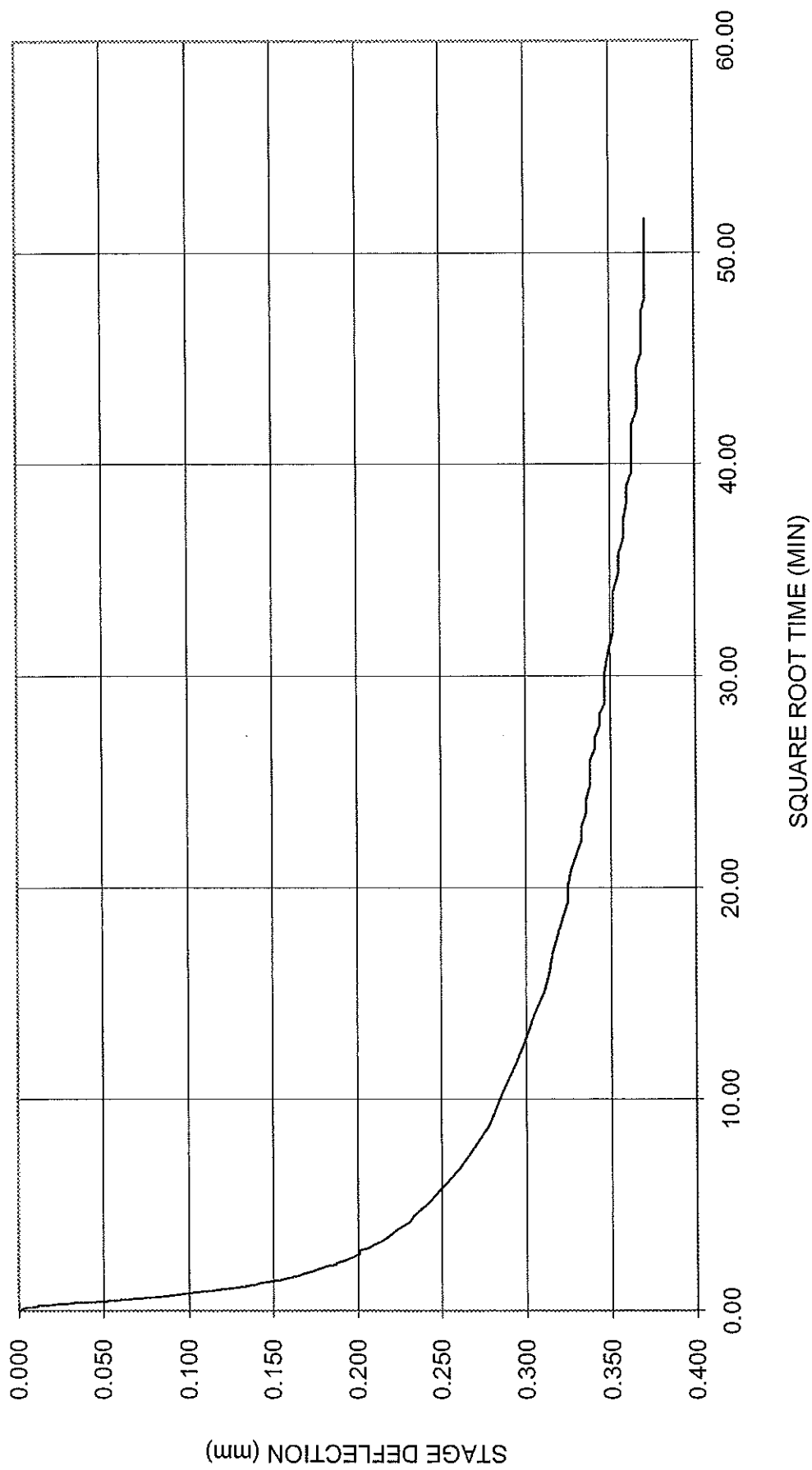


FIGURE B51

# OEDOMETER CONSOLIDATION SUMMARY FIGURE B52

## SAMPLE IDENTIFICATION

Project Number	04-1116-045	Sample Number	ST#2
Borehole Number	18S	Sample Depth, m	38.1-38.7

## TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	06/08/2004		
Date Completed	06/16/2004		

## SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.91	Unit Weight, kN/m <sup>3</sup>	17.36
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	11.95
Area, cm <sup>2</sup>	31.65	Specific Gravity, measured	2.78
Volume, cm <sup>3</sup>	60.45	Solids Height, cm	0.838
Water Content, %	45.21	Volume of Solids, cm <sup>3</sup>	26.51
Wet Mass, g	107.00	Volume of Voids, cm <sup>3</sup>	33.94
Dry Mass, g	73.69	Degree of Saturation, %	98.1

## TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
0.00	1.910	1.281	1.910				
4.70	1.906	1.276	1.908	13	5.94E-02	4.46E-04	2.59E-06
9.54	1.905	1.275	1.906	17	4.53E-02	1.08E-04	4.80E-07
19.29	1.899	1.267	1.902	103	7.45E-03	3.22E-04	2.35E-07
38.71	1.890	1.257	1.895	108	7.05E-03	2.43E-04	1.68E-07
77.48	1.875	1.239	1.883	94	7.99E-03	2.03E-04	1.59E-07
154.67	1.854	1.214	1.865	158	4.66E-03	1.42E-04	6.51E-08
309.42	1.821	1.174	1.838	119	6.02E-03	1.12E-04	6.58E-08
618.54	1.724	1.058	1.773	225	2.96E-03	1.64E-04	4.77E-08
1237.06	1.565	0.869	1.645	540	1.06E-03	1.35E-04	1.40E-08
2473.96	1.461	0.744	1.513	356	1.36E-03	4.40E-05	5.88E-09
1237.06	1.471	0.756	1.466				
309.42	1.508	0.801	1.490				
77.48	1.561	0.864	1.535				
19.29	1.610	0.922	1.586				
4.70	1.662	0.984	1.636				

**Notes:**

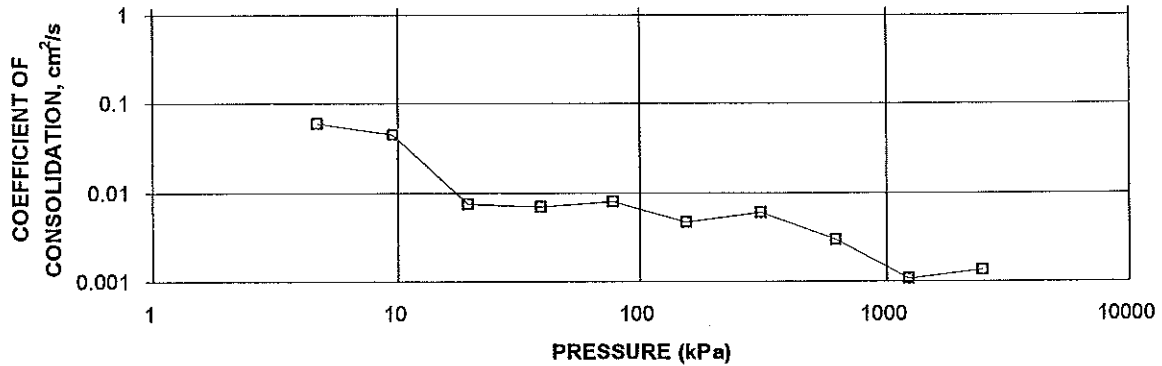
k calculated using cv based on t<sub>90</sub> values.

## SAMPLE DIMENSIONS AND PROPERTIES - FINAL

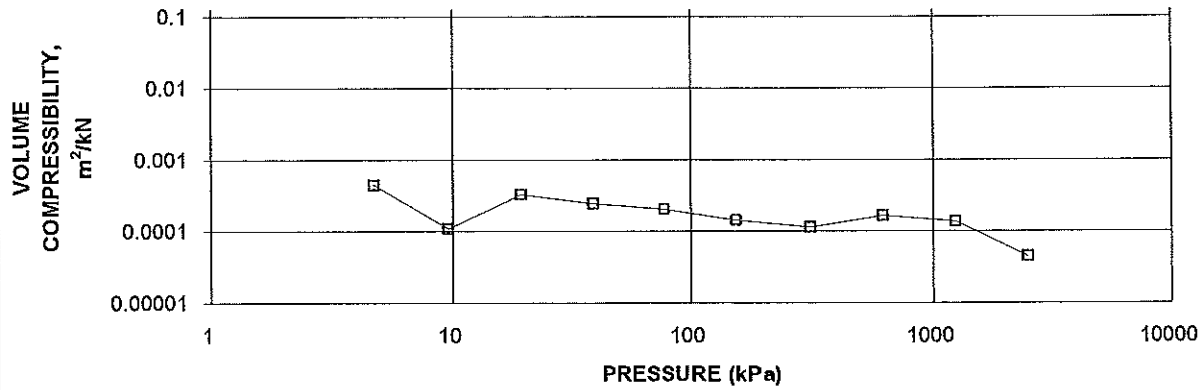
Sample Height, cm	1.66	Unit Weight, kN/m <sup>3</sup>	18.76
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	13.74
Area, cm <sup>2</sup>	31.65	Specific Gravity, measured	2.78
Volume, cm <sup>3</sup>	52.60	Solids Height, cm	0.838
Water Content, %	36.53	Volume of Solids, cm <sup>3</sup>	26.51
Wet Mass, g	100.61	Volume of Voids, cm <sup>3</sup>	26.09
Dry Mass, g	73.69		

# OEDOMETER CONSOLIDATION SUMMARY FIGURE B53

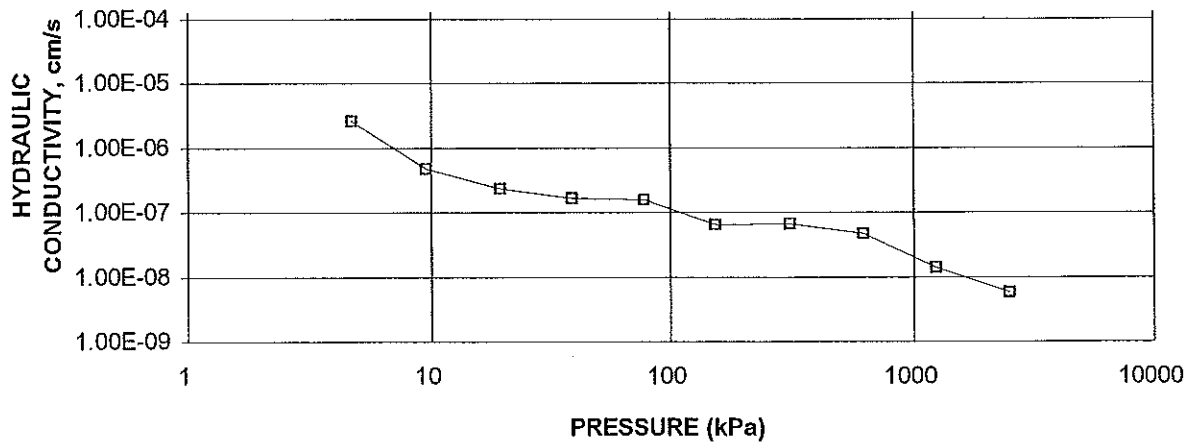
CONSOLIDATION TEST  
CV cm<sup>2</sup>/s VS PRESSURE (kPa)  
BH 18S ST#2



CONSOLIDATION TEST  
MV m<sup>2</sup>/kN vs PRESSURE (kPa)  
BH 18S ST#2



CONSOLIDATION TEST  
HYDRAULIC CONDUCTIVITY vs PRESSURE  
BH 18S ST#2



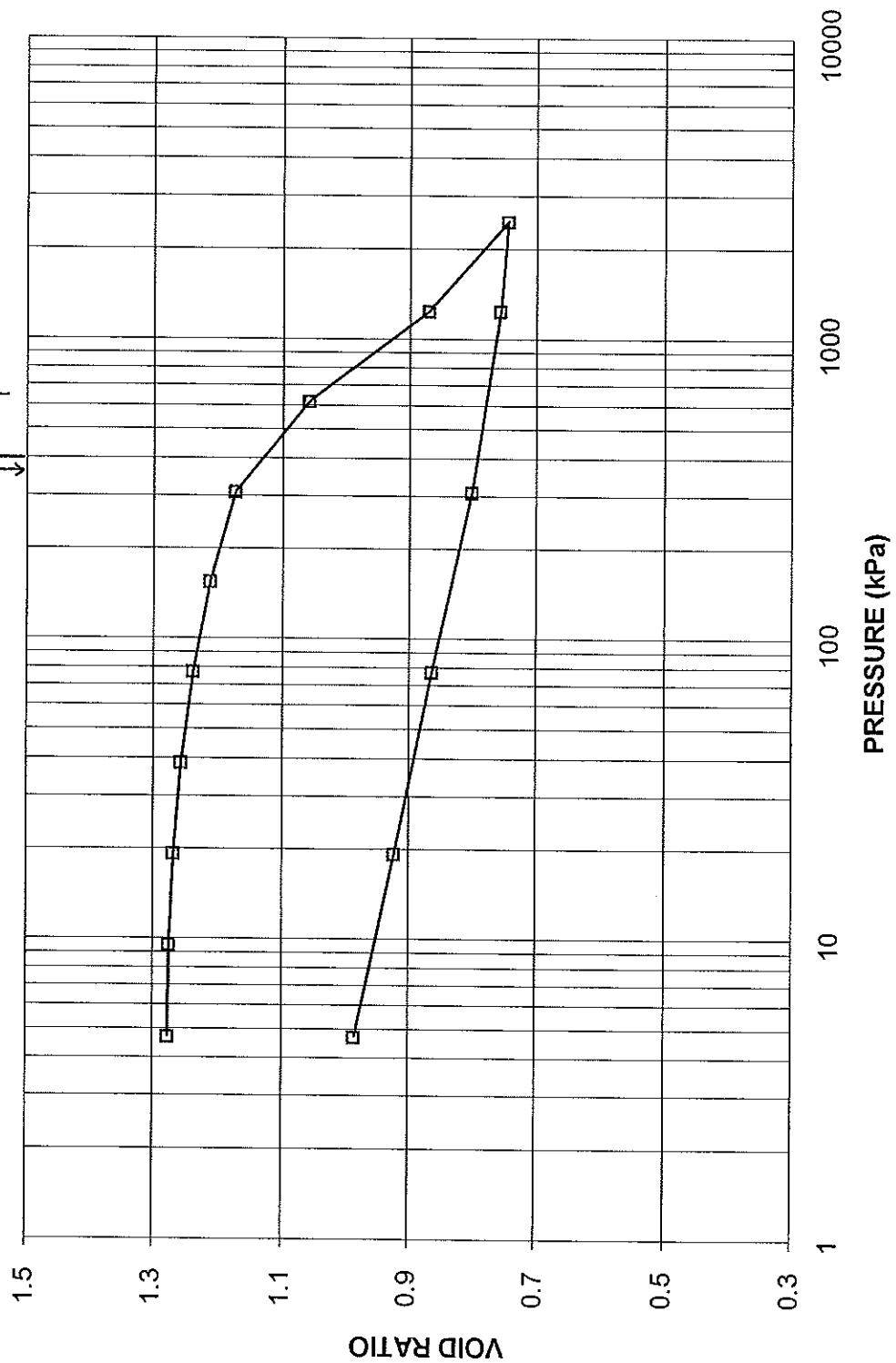


CONSOLIDATION TEST  
VOID RATIO VS. LOG PRESSURE

FIGURE B54

CONSOLIDATION TEST  
VOID RATIO vs PRESSURE  
BH 18S ST#2

$p_0$   
 $p_c$



**Appendix C**  
**Limit Equilibrium Analysis**

Thurber Engineering Ltd. - Toronto  
 15-64-15 (Highway 69, Four Laning, Estaire)  
 SBL, North Abutment Head Slope  
 July 5, 2004  
 Undrained Analysis - H=6m + 3m surcharge (sand)  
 Head slope at 1.5H:1V

	Gamma	C	Phi	Min	Piezo
	kN/m3	kPa	deg	c/p	Surf.
Ballast	19	0	42	0	1
Sand	21.2	0	32	0	1
Rockfill	19	0	42	0	1
Sand & Silt (NP)	19	0	32	0	1
Clayey Silt	19	60	0	0	1
Sand & Silt (NP)	19	0	32	0	1
Clay (CL)	18.5	45	0	0	1
Clay (CH)	17.5	90	0	0	1
Si Clay (CL-ML)	19	80	0	0	1
Silt & Sand (NP)	21	0	32	0	1
Bedrock	(Infinitely Strong)				

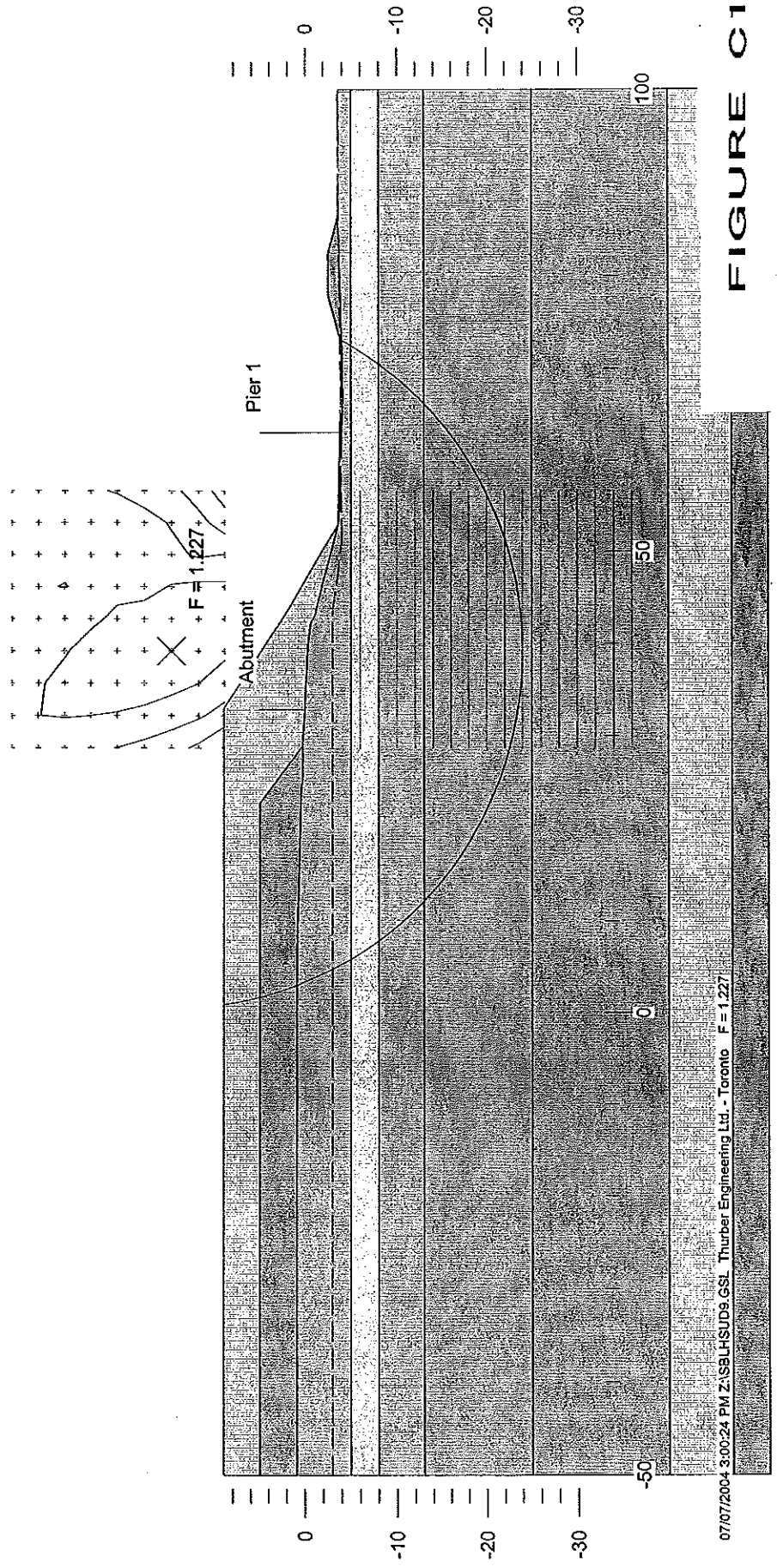


FIGURE C1

	Gamma C kN/m <sup>3</sup>	C kPa	Phi deg	Min c/p	Piezo Surf. 1
Ballast	19	0	42	0	1
Sand	21.2	0	32	0	1
Rockfill	19	0	42	0	1
Sand & Silt (NP)	19	0	32	0	1
Clayey Silt	19	60		.25	2
Sand & Silt (NP)	19	0	32	0	1
Clay (CL)	18.5	45	0	.25	3
Clay (CH)	17.5	90	0	.21	4
Si Clay (CL-ML)	19	80	0	.25	5
Silt & Sand (NP)	21	0	32	0	1
Bedrock	(Infinitely Strong)				

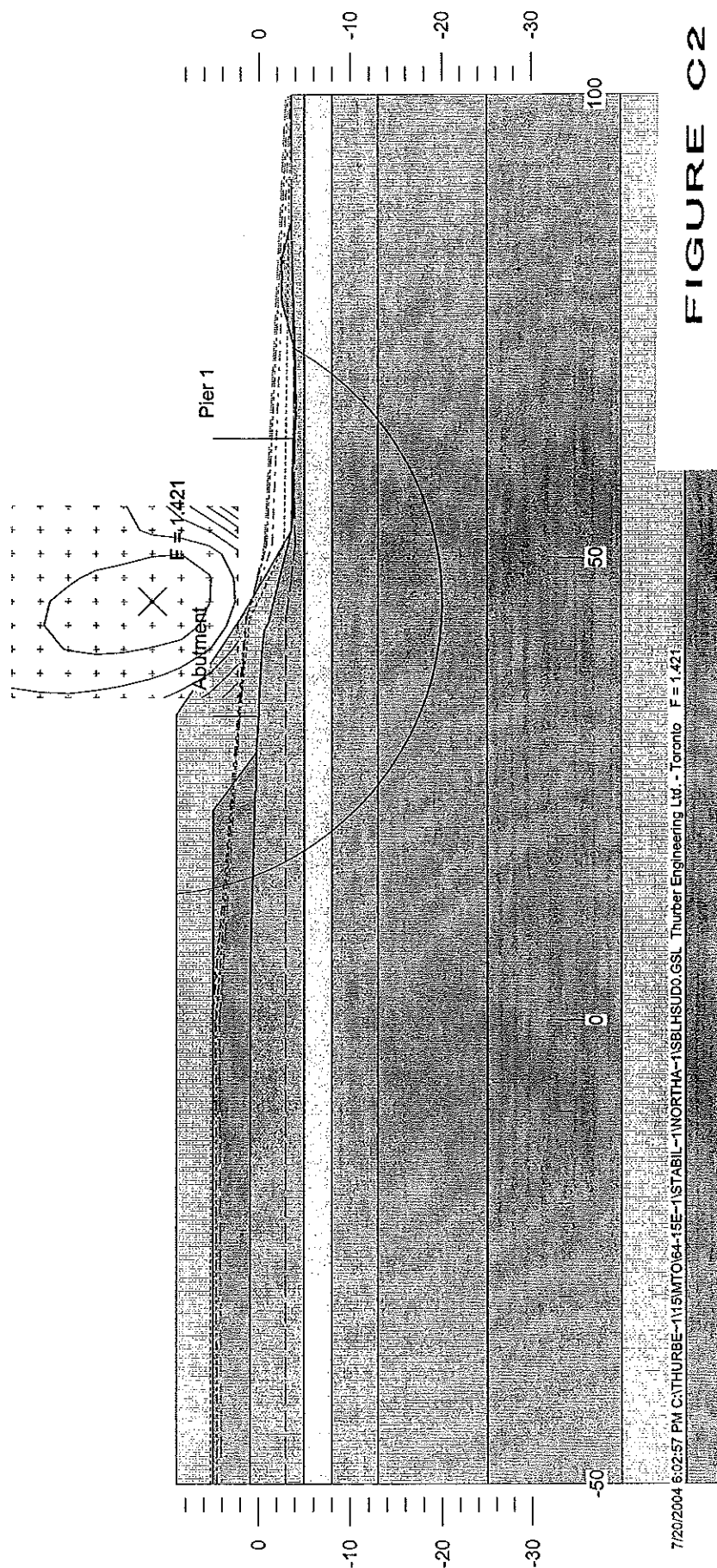
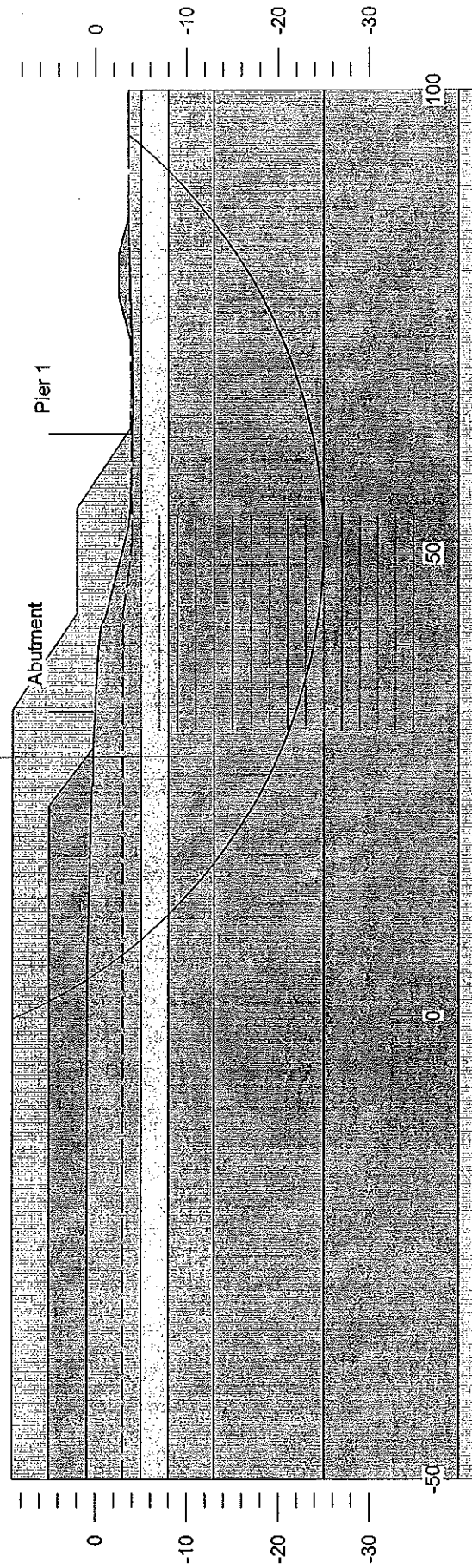
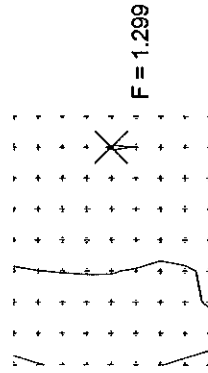


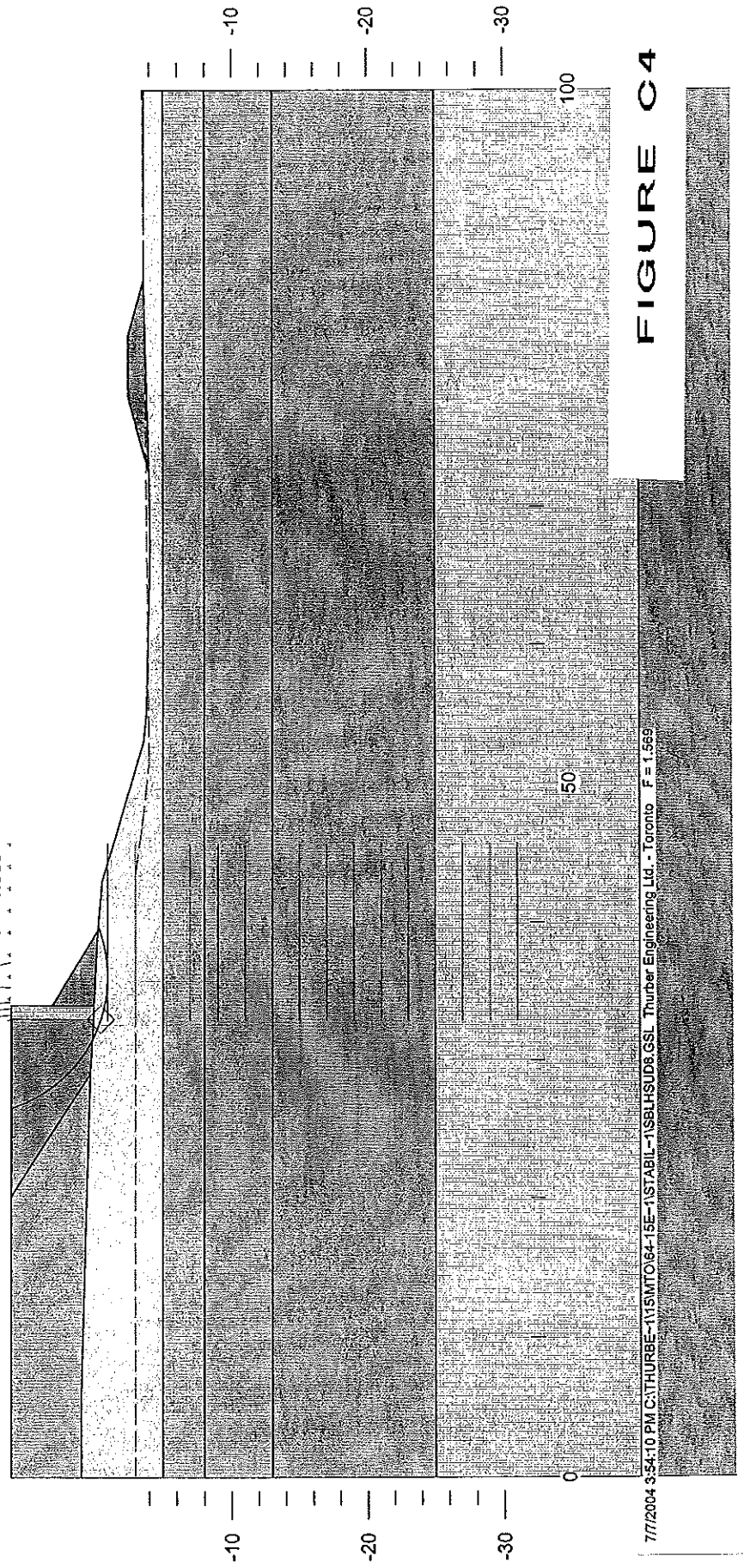
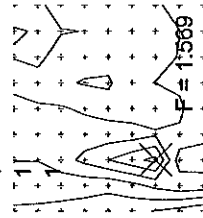
FIGURE C2

Thurber Engineering Ltd. - Toronto  
 15-64-15 (Highway 69, Four Lining, Estaire)  
 SBL, North Abutment Head Slope  
 July 5, 2004  
 Undrained Analysis - H=6m + 3m surcharge  
 Head slope at 1.5H:1V and 11.5m wide berm 5.8m above floodplain elevation

	Gamma	C	Phi	Min	Piezo
	kN/m3	kPa	deg	c/p	Surf.
Ballast	19	0	42	0	1
Sand	21.2	0	32	0	1
Rockfill	19	0	42	0	1
Sand & Silt (NP)	19	0	32	0	1
Clayey Silt	19	60	0	0	1
Sand & Silt (NP)	19	0	32	0	1
Clay (CL)	18.5	45	0	0	1
Clay (CH)	17.5	90	0	0	1
Si Clay (CL-ML)	19	80	0	0	1
Silt & Sand (NP)	21	0	32	0	1
Bedrock	(Infinitely Strong)				



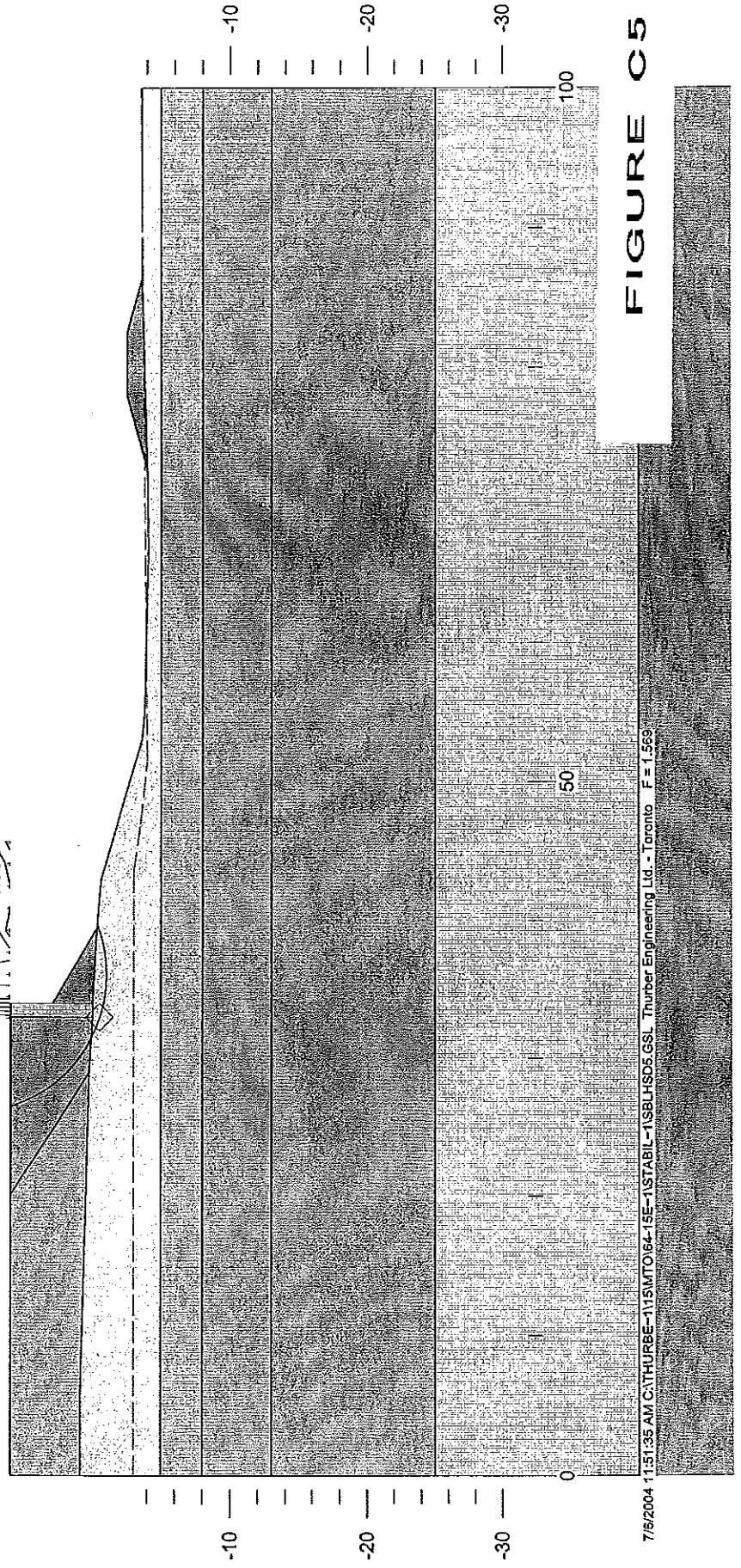
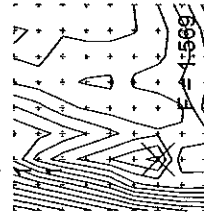
	Gamma C	Phi	Min	Piezo
	kN/m <sup>3</sup>	deg	c/p	Surf.
Rockfill(front)	19	0	42	0
North Abutment	.01	30000	0	0
Sand	22.8	0	35	0
Rockfill(behind)	19	0	42	0
Sand & Silt (NP)	19	0	32	0
Clayey Silt	19	60	0	.25
Sand & Silt (NP)	19	0	32	0
Clay (CL)	18.5	45	0	.25
Clay (CH)	17.5	90	0	.21
Si Clay (CL-ML)	19	80	0	.25
Silt & Sand (NP)	21	0	32	0
Bedrock	(Infinitely Strong)			





Thurber Engineering Ltd. - Toronto  
 15-64-15 (Highway 69, Four Laning, Estaire)  
 SBL, North Abutment Head Slope  
 July 5, 2004  
 Drained Analysis  
 Abutment Final Configuration

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Rockfill(front)	19	0	42	0
North Abutment	.01	30000	0	0
Sand	22.8	0	35	0
Rockfill(behind)	19	0	42	0
Sand & Silt (NP)	19	0	32	0
Clayey Silt	19	0	30	0
Sand & Silt (NP)	19	0	32	0
Clay (CL)	18.5	0	28	0
Clay (CH)	17.5	0	26	0
Si Clay (CL-ML)	19	0	30	0
Silt & Sand (NP)	21	0	32	0
Bedrock	(Infinitely Strong)			



Thurber Engineering Ltd. - Toronto  
 15-64-15 (Highway 69 Four Lining, Estaire)  
 10+580, SBL, Embankment Slope  
 July 6, 2004  
 Undrained Analysis  
 Rockfill embankment with 3m surcharge (total H=9m)

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Sand	21.2	35	0	1
Rockfill	19	42	0	1
Sand & Silt (NP)	19	32	0	1
Clayey Silt	19	60	0	1
Sand & Silt (NP)	19	32	0	1
Clay (CL)	18.5	45	0	1
Clay (CH)	17.5	90	0	1
Si Clay (CL-ML)	19	80	0	1
Silt & Sand (NP)	21	32	0	1
Bedrock	(Infinitely Strong)			

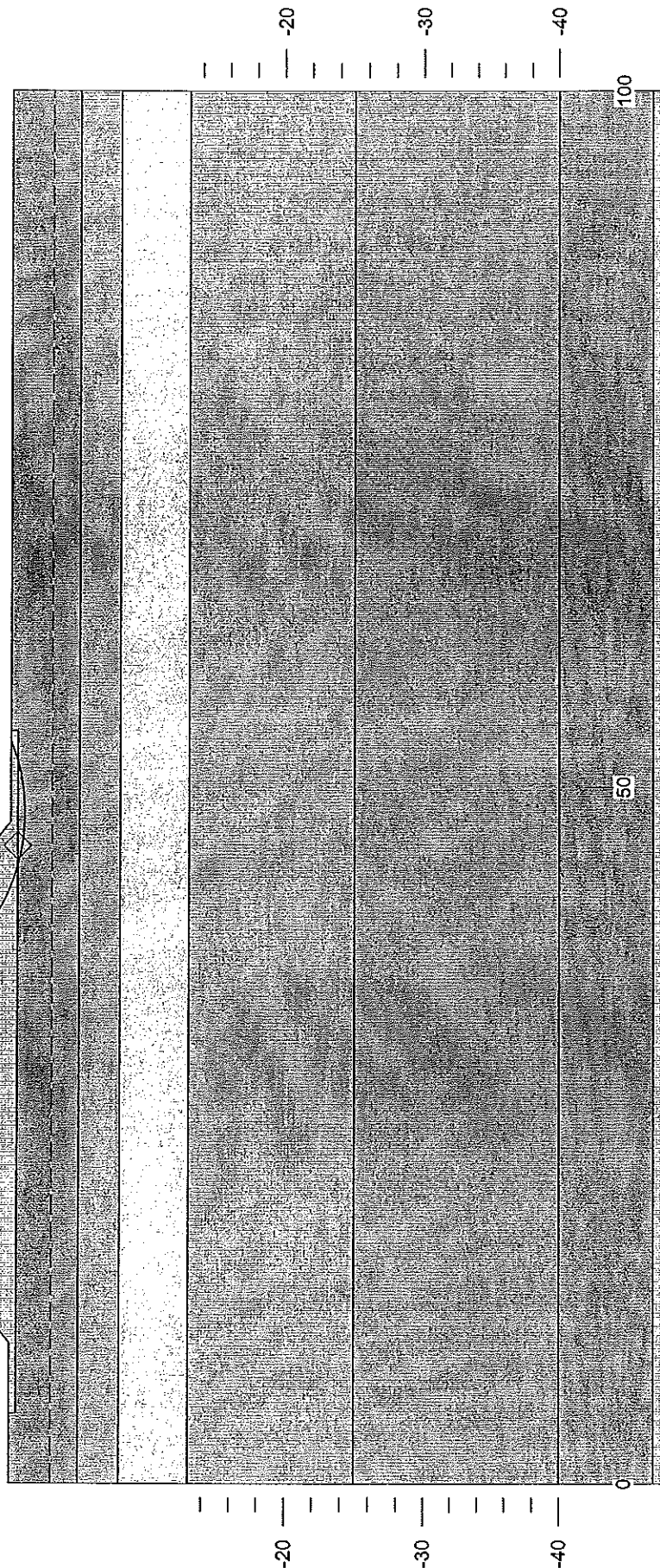
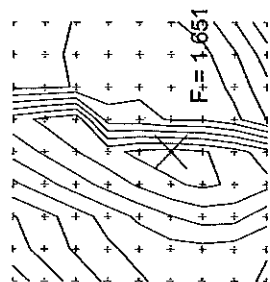
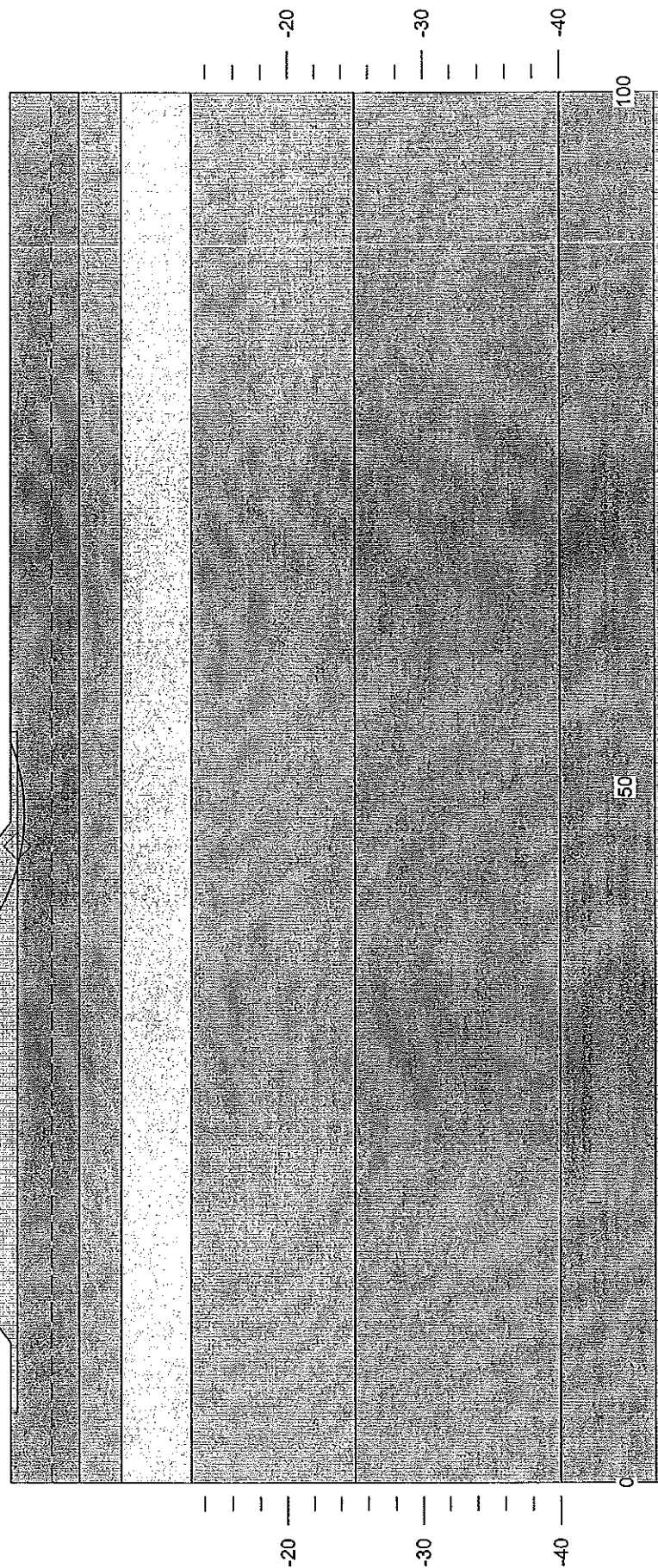
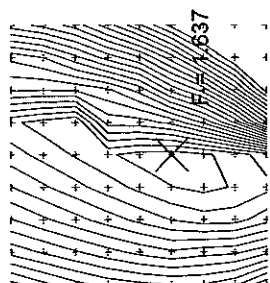


FIGURE C6

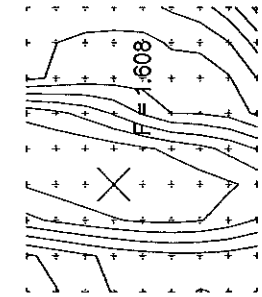


Thurber Engineering Ltd. - Toronto  
 15-64-15 (Highway 69 Four Lining, Estaire)  
 10+580, SBL, Embankment Slope  
 July 6, 2004  
 Drained Analysis  
 Rockfill embankment with 3m surcharge ( total H=9m)

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Sand	21.2	0	32	0
Rockfill	19	0	42	0
Sand & Silt (NP)	19	0	32	0
Clayey Silt	19	0	30	0
Sand & Silt (NP)	19	0	32	0
Clay (CL)	18.5	0	28	0
Clay (CH)	17.5	0	26	0
Si Clay (CL-ML)	19	0	30	0
Silt & Sand (NP)	21	0	32	0
Bedrock	(Infinitely Strong)			



Thurber Engineering Ltd. - Toronto  
 15-64-15 (Highway 69 Four Lining, Estaire)  
 10+580, SBL, Embankment Slope  
 July 6, 2004  
 Undrained Analysis  
 Sand fill embankment with 3m surcharge (Total H=9m)



	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Sand	21.2	0	0	1
Sand & Silt (NP)	19	0	0	1
Clayey Silt	19	60	0	1
Sand & Silt (NP)	19	0	0	1
Clay (CL)	18.5	45	0	1
Clay (CH)	17.5	90	0	1
Si Clay (CL-ML)	19	80	0	1
Silt & Sand (NP)	21	0	0	1
Bedrock	(Infinitely Strong)			

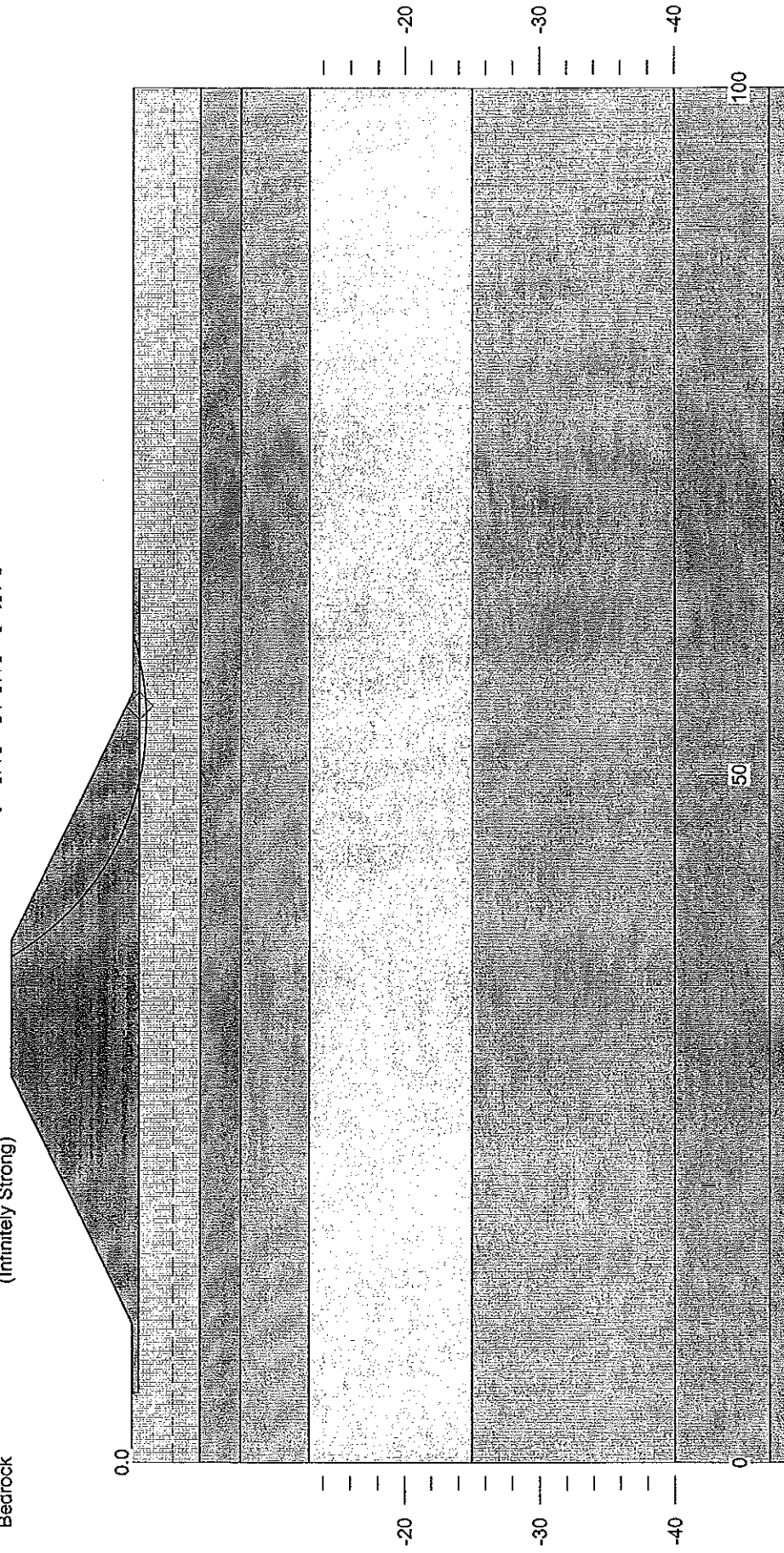


FIGURE C8

Thurber Engineering Ltd. - Toronto  
 15-64-15 (Highway 69 Four Lining, Estaire)  
 10+580, SBL, Embankment Slope  
 July 6, 2004  
 Drained Analysis  
 Sand fill embankment with 3m surcharge (Total H=9m)

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Sand	21.2	0	0	1
Sand & Silt (NP)	19	0	32	1
Clayey Silt	19	0	30	1
Sand & Silt (NP)	19	0	32	1
Clay (CL)	18.5	0	28	1
Clay (CH)	17.5	0	26	1
Si Clay (CL-ML)	19	0	30	1
Silt & Sand (NP)	21	0	32	1
Bedrock	(Infinitely Strong)			

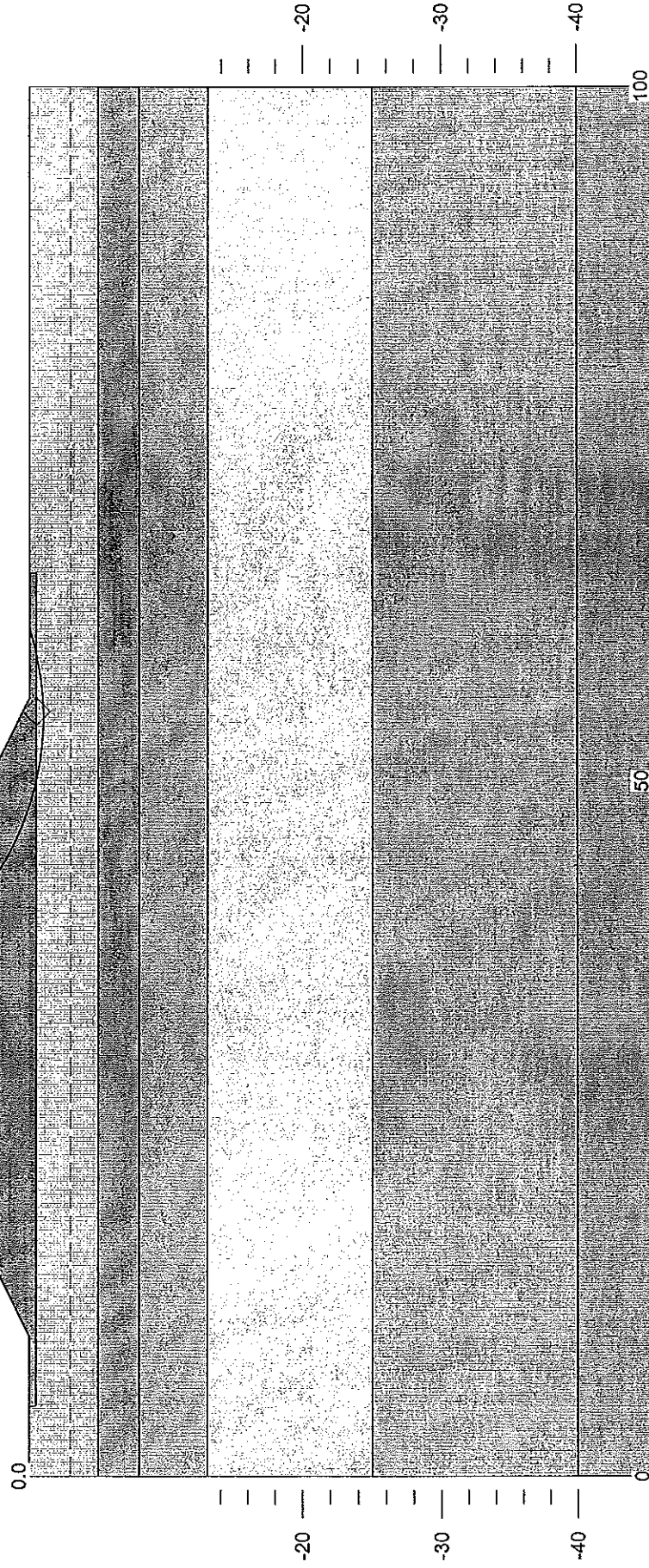
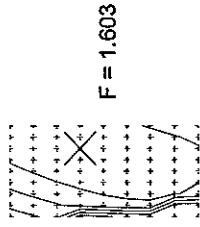


FIGURE C9

Thurber Engineering Ltd. - Toronto  
 15-64-15 (Highway 69, Four Laning, Estaire)  
 NBL, North Abutment Head Slope  
 June 17, 2004  
 Undrained Analysis  
 H= 6.3 rockfill +3m surcharge (sand)

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Ballast	19	0	0	1
Sand	21.2	0	0	1
Rockfill(behind)	19	0	0	1
Sand & Silt (NP)	19	0	0	1
Clay (CL)	18.5	45	0	1
Clay (CH)	17.5	90	0	1
Si Clay (CL-ML)	19	80	0	1
Silt & Sand (NP)	21	0	0	1
Bedrock	(Infinitely Strong)			

$$F_t = 1.305$$

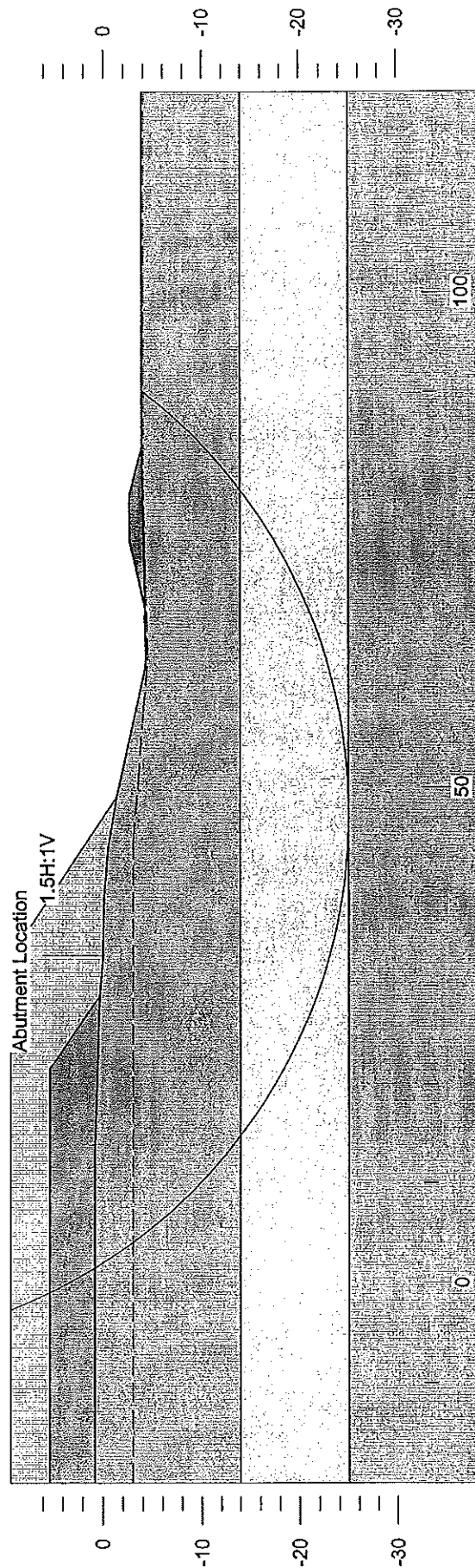
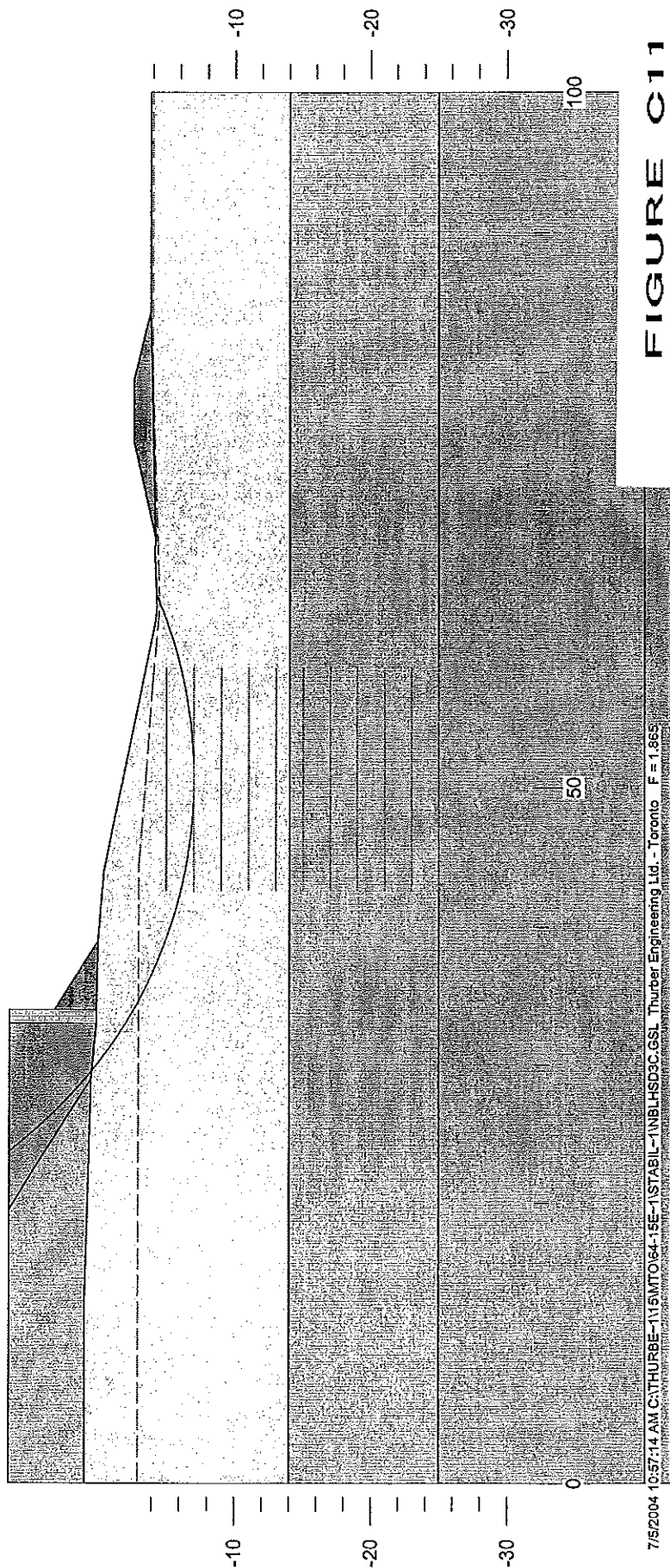


FIGURE C10



Thurber Engineering Ltd. - Toronto  
 15-64-15 (Highway 69, Four Laning, Estaire)  
 NBL, North Abutment Head Slope  
 July 5, 2004  
 Undrained Analysis  
 Abutment Construction in One Stage

	Gamma C	Phi	Min	Piezo
	kN/m <sup>3</sup>	deg	c/p	Surf.
Rockfill(front)	19	0	42	0
North Abutment	.01	30000	0	0
Sand	22.8	0	35	0
Rockfill(back)	19	0	42	0
Sand & Silt (NP)	19	0	32	0
Clay (CL)	18.5	45	0	.25
Clay (CH)	17.5	90	0	.21
Si Clay (CL-ML)	19	80	0	.25
Silt & Sand (NP)	21	0	32	0
Bedrock	(Infinitely Strong)			



Thurber Engineering Ltd. - Toronto  
 15-64-15 (Highway 69, Four Lining, Estaire)  
 NBL, North Abutment Head Slope  
 July 5, 2004  
 Drained Analysis  
 Abutment Long Term Stability

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Rockfill(front)	19	0	42	1
North Abutment	.01	30000	0	1
Sand	22.8	0	35	1
Rockfill(behind)	19	0	42	1
Sand & Silt (NP)	19	0	32	1
Clay (CL)	18.5	0	28	2
Clay (CH)	17.5	0	26	3
Si Clay (CL-ML)	19	0	30	4
Silt & Sand (NP)	21	0	32	5
Bedrock	(Infinitely Strong)			

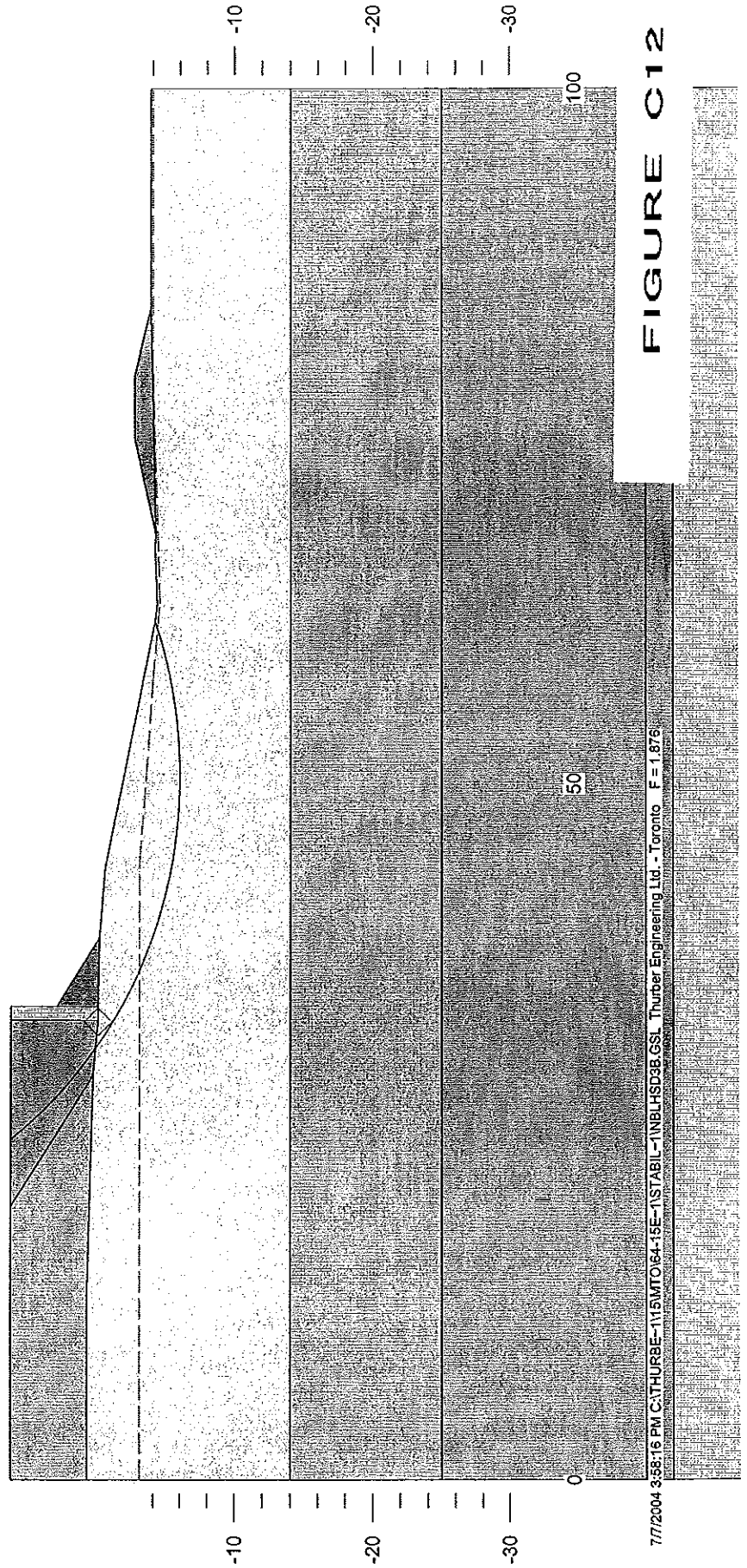
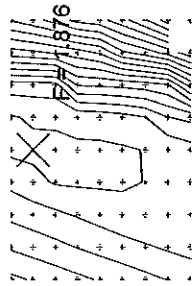


FIGURE C12

Thurber Engineering Ltd. - Toronto  
 15-64-15 (Highway 69 Four Lining, Estaire)  
 10+590, NBL, Embankment Slope  
 July 5, 2004  
 Undrained Analysis - One Stage Construction  
 Rockfill embankment (H=6.3m) with 3m surcharge (Sand)

	Gamma C	Phi	Min	Piezo
	kN/m <sup>3</sup>	deg	c/p	Surf.
Sand	21.2	0	32	0
Rockfill	19	0	42	0
Sand & Silt (NP)	19	0	32	0
Clay (CL)	18.5	45	0	0
Clay (CH)	17.5	90	0	0
Si Clay (CL-ML)	19	80	0	0
Silt & Sand (NP)	21	0	32	0
Bedrock	(Infinitely Strong)			

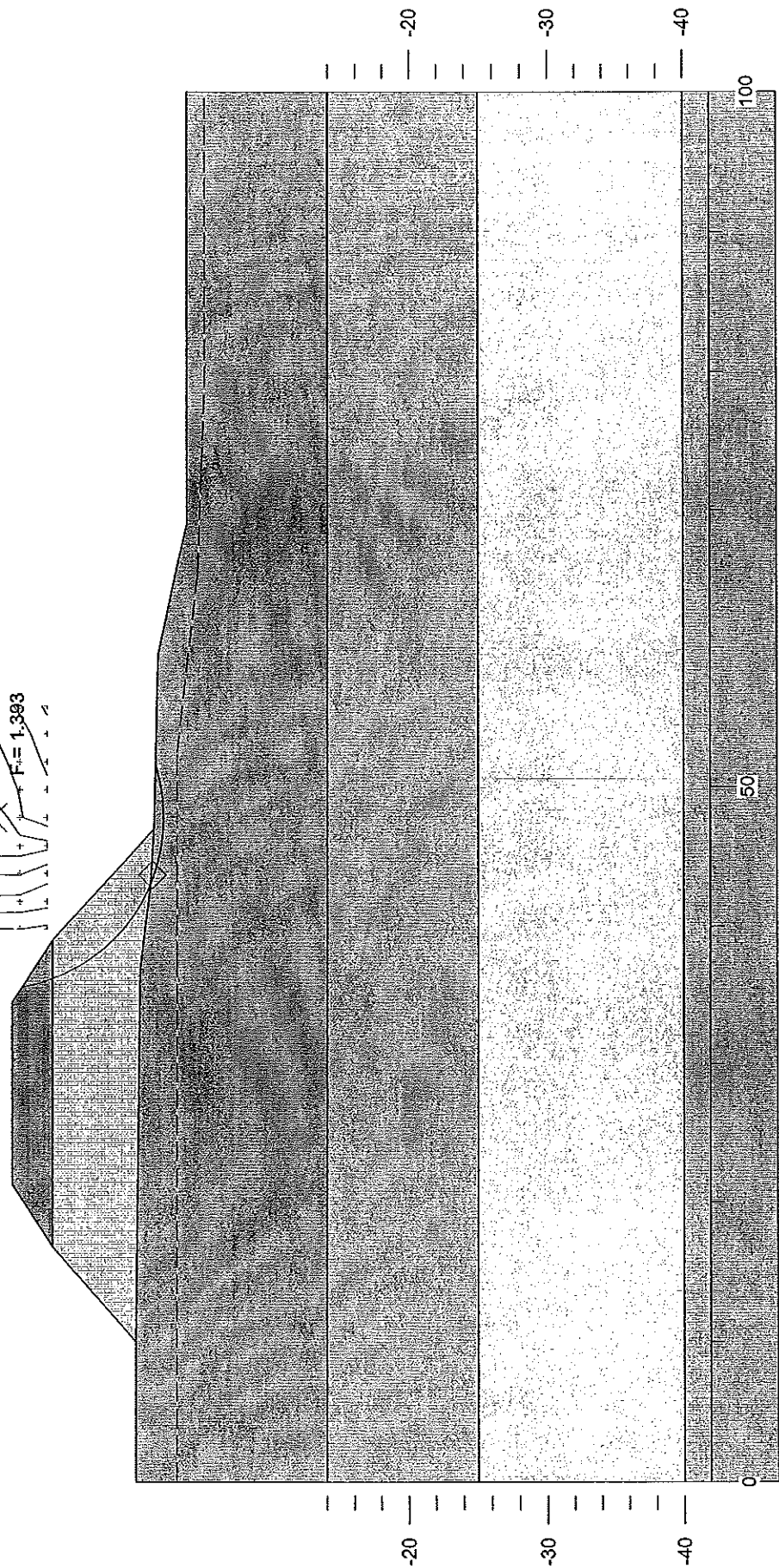
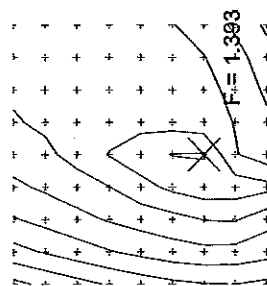


FIGURE C13

Thurber Engineering Ltd. - Toronto  
 15-64-15 (Highway 69 Four Lining, Estaire)  
 10+590, NBL, Embankment Slope  
 July 5, 2004  
 Drained Analysis - Long Term Stability  
 Rockfill embankment (H=6.3m)

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Rockfill	19	0	42	0
Sand & Silt (NP)	19	0	32	0
Clay (CL)	18.5	0	28	0
Clay (CH)	17.5	0	26	0
Si Clay (CL-ML)	19	0	30	0
Silt & Sand (NP)	21	0	32	0
Bedrock	(Infinitely Strong)			

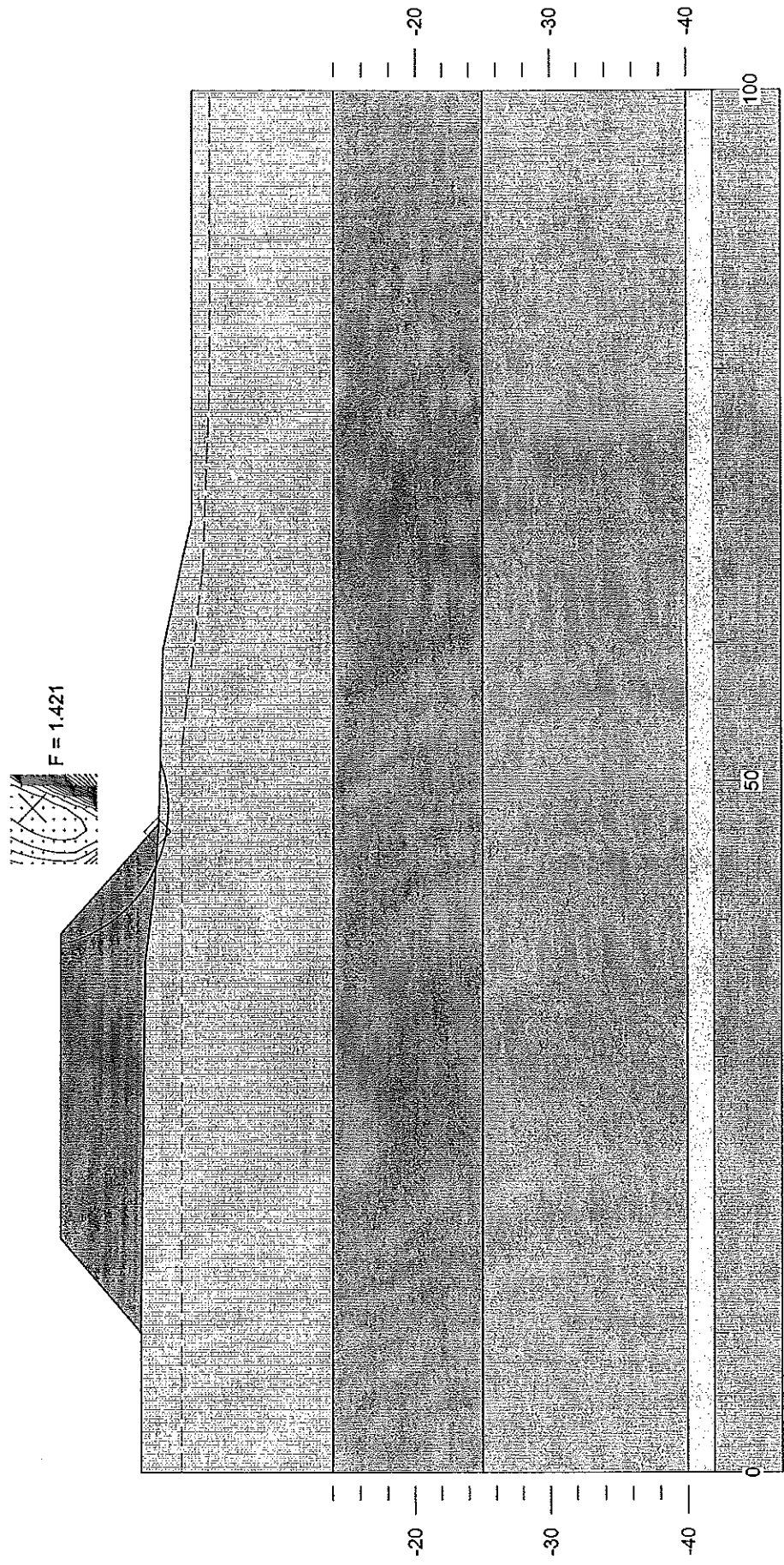


FIGURE C14



Thurber Engineering Ltd. - Toronto  
 15-64-15 (Highway 69 Four Lining, Estaire)  
 10+590, NBL, Embankment Slope  
 June 17, 2004  
 Undrained Analysis

Sand  
 Sand & Silt (NP)  
 Clay (CL)  
 Clay (CH)  
 Si Clay (CL-ML)  
 Silt & Sand (NP)  
 Bedrock

Gamma C	Phi	Min	Piezo
kN/m <sup>3</sup>	deg	c/p	Surf.
21.2	0	0	1
19	0	0	1
18.5	45	0	1
17.5	90	0	1
19	80	0	1
21	0	0	1
(Infinitely Strong)			

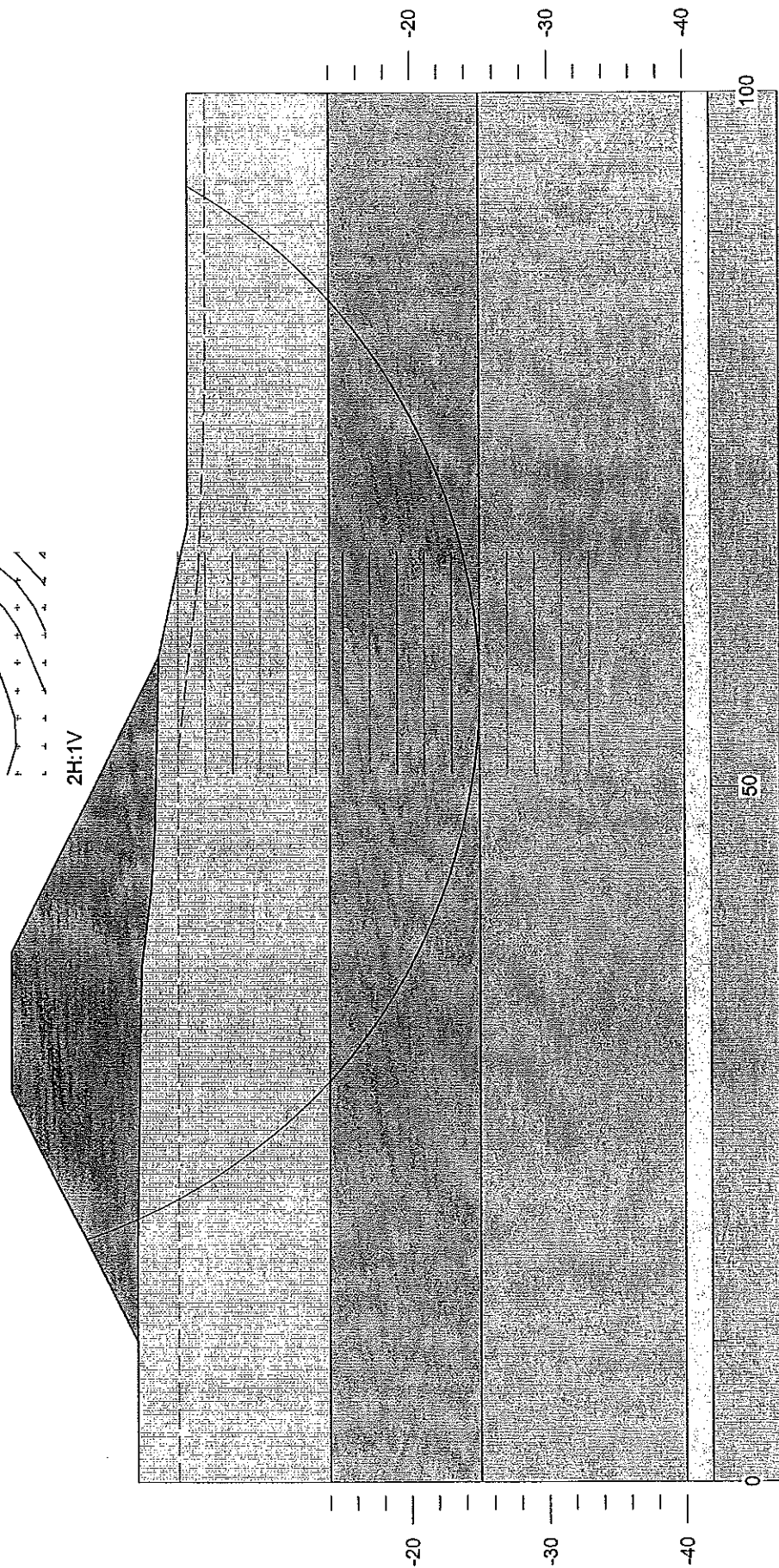
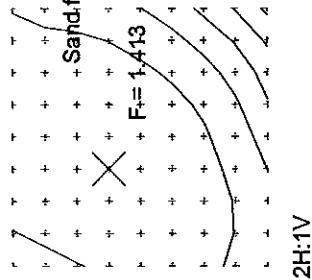
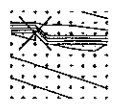
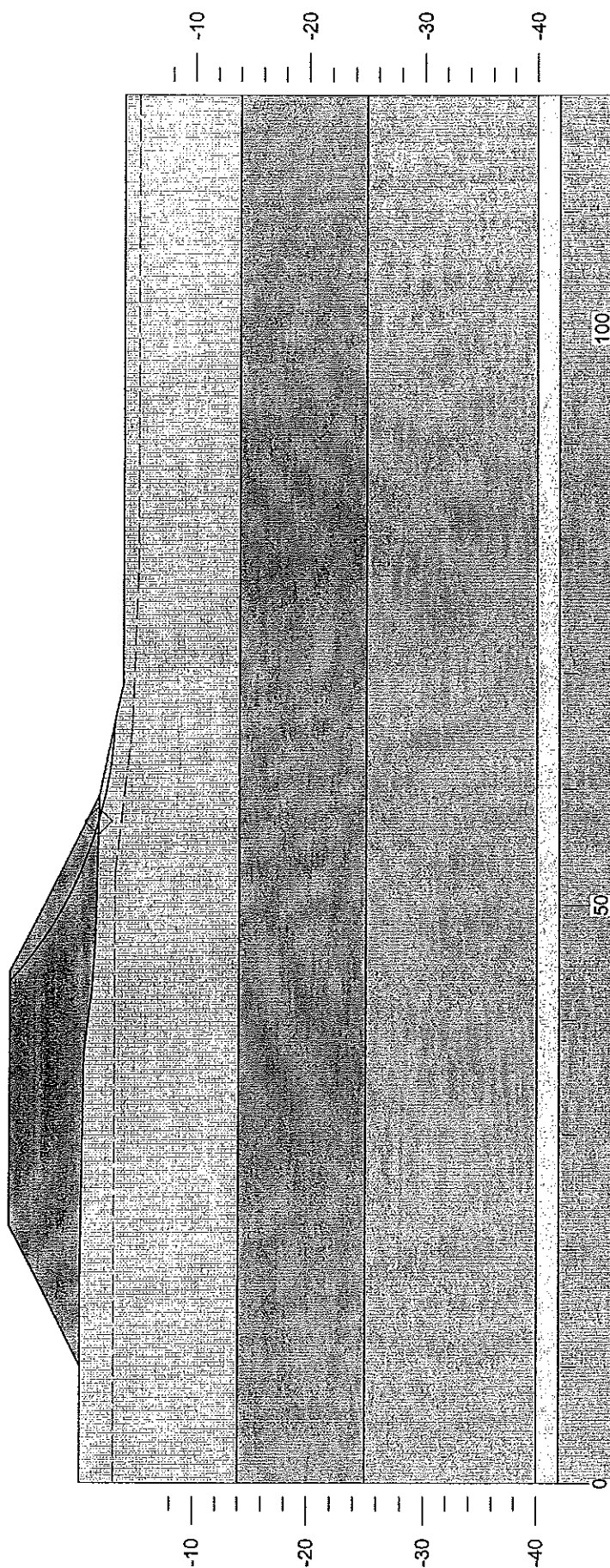


FIGURE C15

Thurber Engineering Ltd. - Toronto  
 15-64-15 (Highway 69 Four Lining, Estaire)  
 10+590, NBL, Embankment Slope  
 June 17, 2004  
 Drained Analysis - Long Term Stability  
 SF = 1.375; bankment near North Abutment with surcharge (Sand) H=6.3m



	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Sand	21.2	0	0	1
Sand & Silt (NP)	19	0	0	1
Clay (CL)	18.5	0	0	1
Clay (CH)	17.5	0	0	1
Si Clay (CL-MIL)	19	0	0	1
Silt & Sand (NP)	21	0	0	1
Bedrock	(Infinitely Strong)			



Thurber Engineering Ltd. - Toronto  
 15-64-15  
 SBL South Abutment Head Slope  
 July 20, 2004  
 Without Footing Load

✕ F = 1.474

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Rockfill	19	0	42	0
Overburden	18	0	30	0
Rockfill/BR INTF	19	0	30.98	0
Gran.A/BR INTF	22.8	0	25	0
Bedrock (BR)	(Infinitely Strong)			

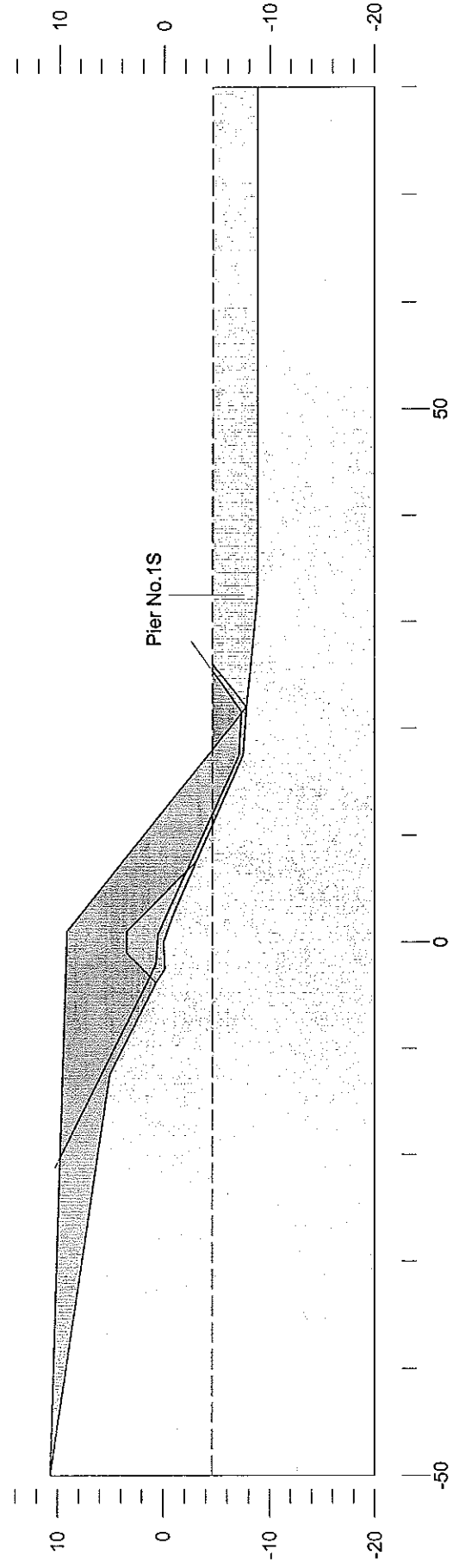


FIGURE C17

Thurber Engineering Ltd. - Toronto  
 15-64-15  
 SBL South Abutment Head Slope  
 July 20, 2004  
 With Footing Load

	Gamma C	Phi	Min	Piezo
	kN/m <sup>3</sup>	deg	c/p	Surf.
Rockfill	19	0	42	0
Overburden	18	0	30	0
Rockfill/BR INTF	19	0	30.98	0
Footing	350	30000	0	0
Gran.A/BR INTF	22.8	0	25	0
Bedrock (BR)	(Infinitely Strong)			

✕ F = 1.551

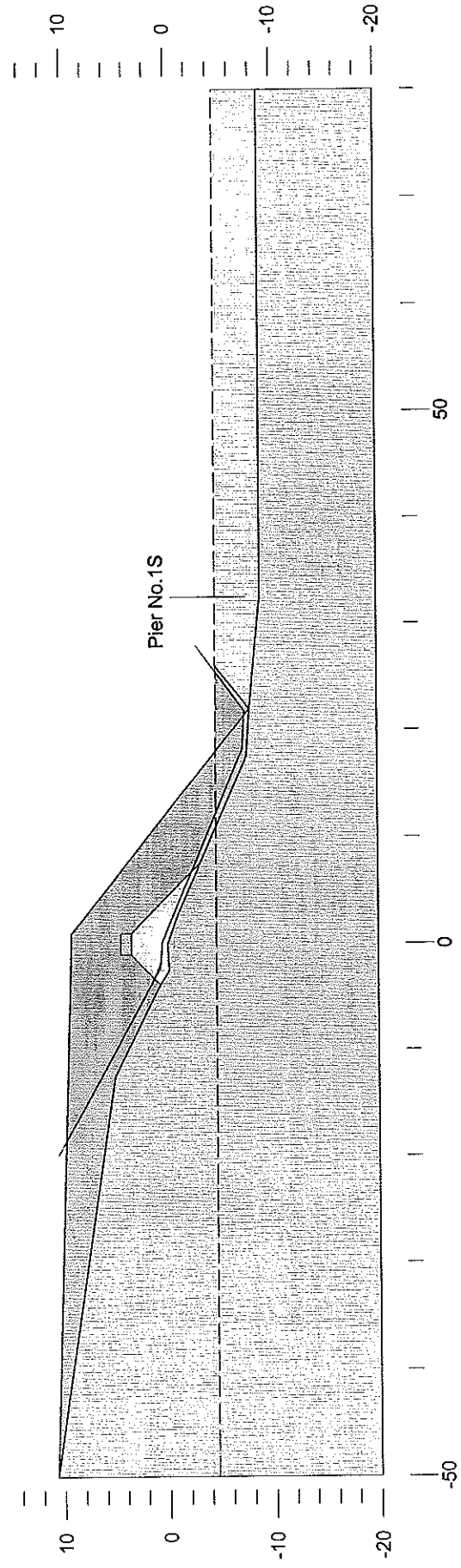


FIGURE C18

Thurber Engineering Ltd. - Toronto  
 15-64-15  
 NBL South Abutment Head Slope  
 July 20, 2004  
 Without Footing Load

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Rockfill	19	42	0	1
Overburden	18	30	0	1
Rockfill/BR INTF	42	30.98	0	1
Gran.A/BR INTF	22.8	25	0	1
Bedrock (BR)	(Infinitely Strong)			

✕ F = 1.455

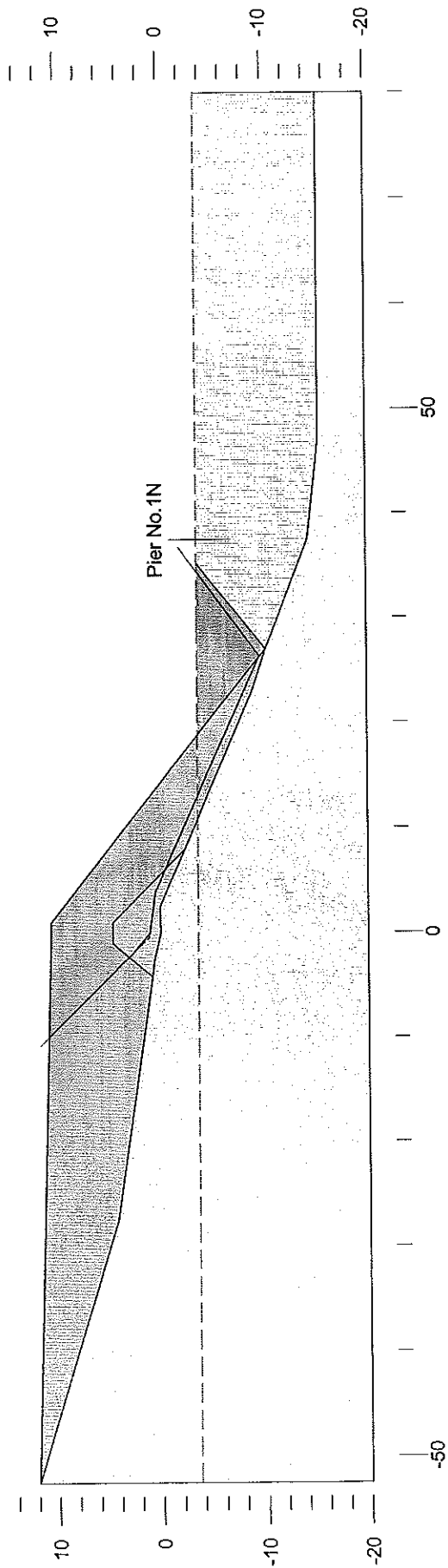


FIGURE C19

Thurber Engineering Ltd. - Toronto  
 15-64-15  
 NBL South Abutment Head Slope  
 July 20, 2004  
 With Footing Load

	Gamma C	Phi	Min	Piezo
	kN/m <sup>3</sup>	deg	c/p	Surf.
Rockfill	19	0	42	0
Overburden	18	0	30	0
Rockfill/BR INTF	19	0	30.98	0
Footing	350	30000	0	0
Gran.A/BR INTF	22.8	0	25	0
Bedrock (BR)	(Infinitely Strong)			

✕ F = 1.801

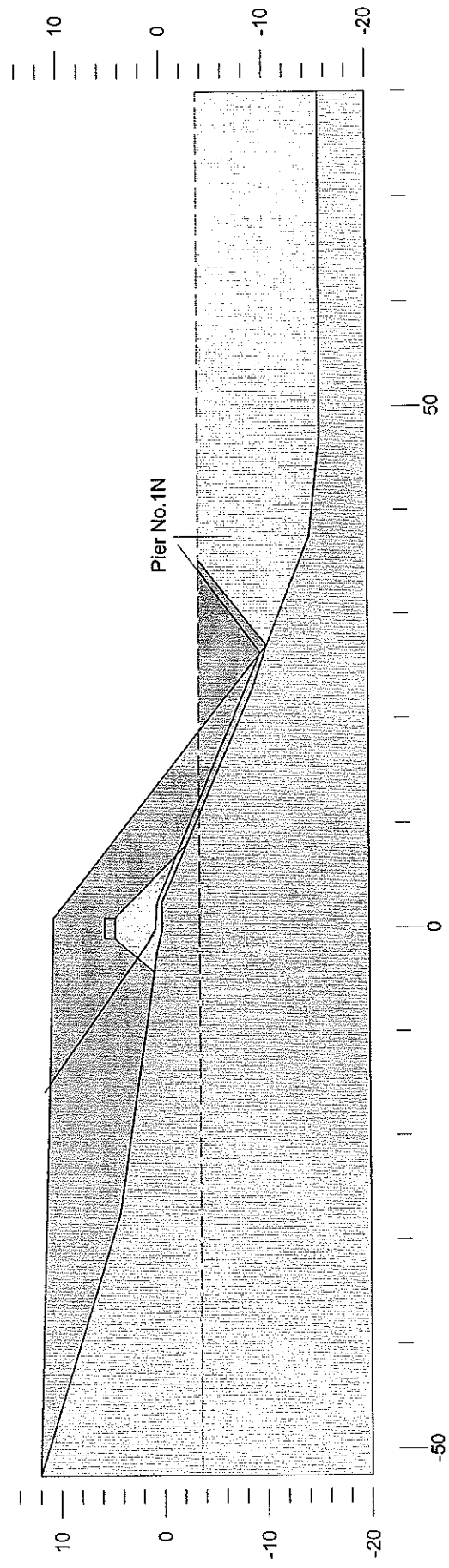


FIGURE C20

**FOUNDATION INVESTIGATION  
AND DESIGN REPORT  
HIGHWAY 69, FOUR-LANING  
FROM 4KM SOUTH OF ESTAIRE TO 1KM NORTH OF HIGHWAY 537, 12KM  
CN RAIL OVERHEAD STRUCTURES AND APPROACH EMBANKMENTS  
ONTARIO**

**G.W.P. 312-99-00  
WP 5047-00-01, SITE 46-494N  
WP 5048-00-01, SITE 46-494S  
Geocres Number: 41I-184  
VOLUME 2/2**

**Report to**

**Ministry of Transportation Ontario  
Planning and Design Section, North Bay**

**Thurber Engineering Ltd.  
2010 Winston Park Drive, Suite 103  
Oakville, Ontario  
L6H 5R7  
Phone: (905) 829 8666  
Fax: (905) 829 1166**

**September 7, 2004**

**C:\Thurber Files\15\MTO\64-15 Estaire\64-15-Final Report-September-2004.doc**



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Sheet 2

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Sheet 6

Sheet 7

Test Hole Location Plan

SBL - Borehole Locations and Soil Strata

SBL – Cross Sections

NBL - Borehole Locations and Soil Strata

NBL – Cross Sections

SBL – North Approach Embankment - Borehole Locations and  
Soil Strata

NBL – North Approach Embankment - Borehole Locations and  
Soil Strata

North Approach Embankments – Cross Sections

## **Appendices**

Appendix A

Appendix B

Appendix C

Record of Borehole Sheets

Laboratory Test Results

Slope Stability Analysis Results

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ConeTec, Inc.'s Data Report

ConeTec, Inc.'s Seismic Analysis Report

Primary Consolidation Analysis Results

Wick Drain Analysis Results

Secondary Consolidation Analysis Results

Non-Standard Special Provisions

Comparison of Design Alternatives

## Appendix D

### ConeTec Inc. Report



## Field Data Report

# ConeTec Investigations Ltd.



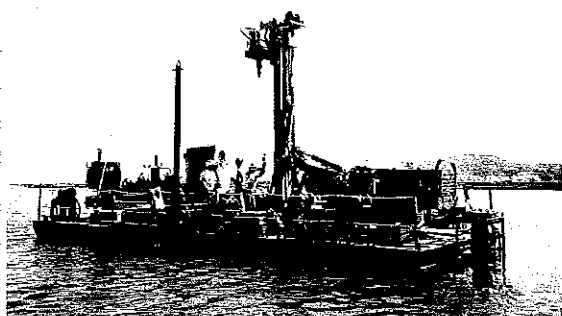
9113 Shaughnessy Street  
Vancouver, BC V6P 6R9

Tel: 604-327-4311

Fax: 604-327-4066

Toll Free: 800-567-7969

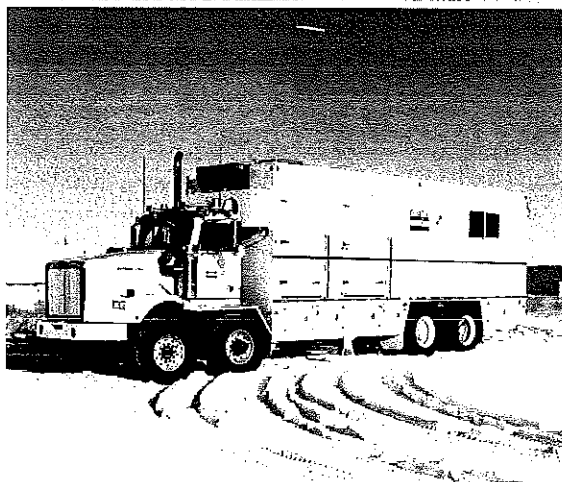
Email: [insitu@conetec.com](mailto:insitu@conetec.com)



*Prepared for:*

Thurber Engineering Ltd.  
Highway 69-4 Lane  
Estaire, ON  
Job No: 04-166

- Mar 15-18, 2004 -





## Cone Penetration Tests (CPTU)

The cone penetration tests (CPTU) with pore pressure measurement were carried out by ConeTec using an integrated electronic cone system.

A 20 ton compression type cone (refer to Figure CPTU) was used for all of the soundings. This cone has a tip area of 15 sq. cm. and a friction sleeve area of 225 sq. cm. The compression cone is designed with an equal end area friction sleeve and a tip end area ratio of approximately 0.85. A porewater pressure filter was located directly behind the cone tip. The filter was made of porous plastic and was 5.0 mm thick. Each of the porewater pressure filters was saturated under vacuum pressure prior to penetration. Porewater pressure dissipation data was recorded at 5-second intervals during pauses in penetration as directed by the field representative.

The cone was capable of recording the following parameters at varying depth intervals:

- Tip Resistance ( $q_c$ )
- Sleeve Friction ( $f_s$ )
- Penetration Pore Pressure ( $u$ )
- Temperature ( $T$ )
- Cone Inclination ( $I$ )

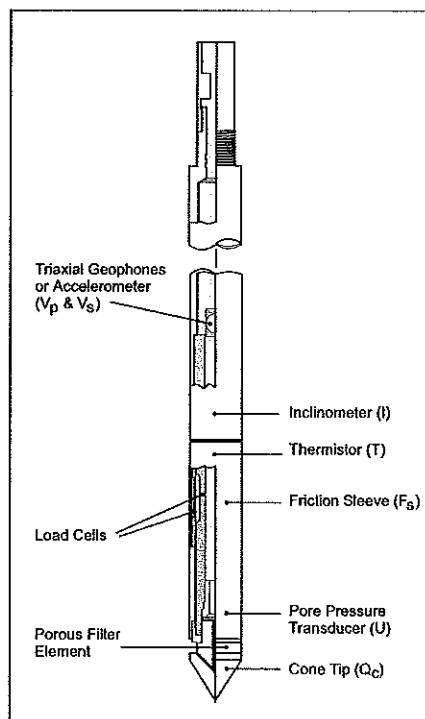


Figure - CPTU

A summary of the cone penetration tests carried out is presented in Table CPTU (Appendix CPTU).

Selected parameters were printed simultaneously on a printer and stored on a floppy disk for future analysis and reference. All cone penetration testing was carried out in accordance with ASTM D-5778-95.

A complete set of baseline readings was taken prior to and at the completion of each sounding to determine temperature shifts and any zero load offsets. Corrections for temperature shifts and zero load offsets can be extremely important, especially when the recorded loads are relatively small. In sandy soils, however, these corrections are generally negligible. Graphical plots of all CPT data are presented in Appendix CPTU.

The inferred stratigraphic profile at each CPT test location is included with this report. The stratigraphic interpretations are based on relationships between corrected cone tip resistance,  $q_t$ , sleeve friction,  $f_s$ . The friction ratio,  $R_f$  ( $100 \times f_s/q_t$ ), is a calculated parameter which is used to identify the type of soil and hence gives an indication of its behavior. Generally, soft cohesive soils have high friction ratios, low cone bearing pressures and generate large porewater pressures during penetration. Cohesionless soils have lower friction ratios, high cone bearing pressures and generate little in the way of excess porewater pressure during penetration. The classification of soils is based on correlations summarized by Robertson (1990), as shown in Figure SBT. It is not always possible to clearly identify a soil type based on  $q_t$  and  $f_s$  alone. Experience, judgment and analyses of porewater pressure generation during penetration and subsequent dissipation tests should be used in arriving at the soil type in these ambiguous situations.

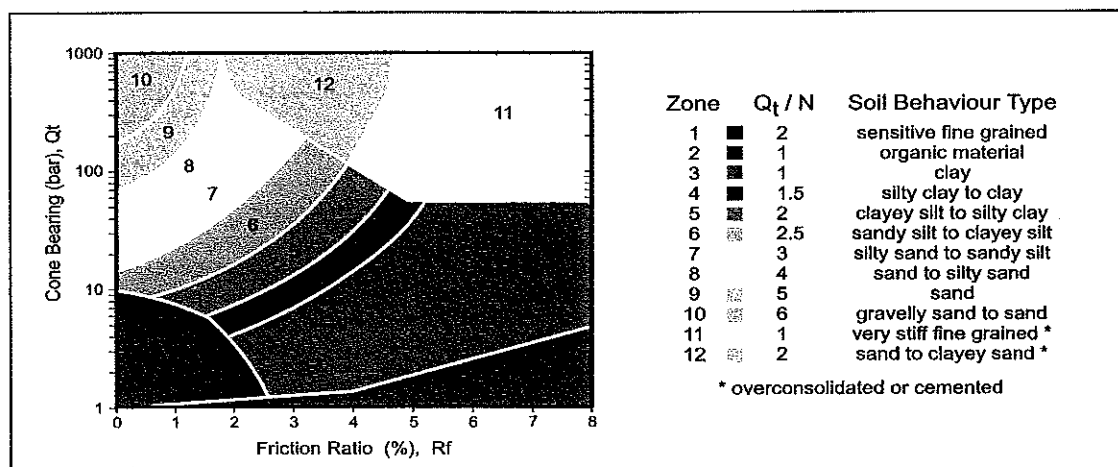


Figure SBT – Non-Normalized Soil Behavior Type Chart, Robertson (1990)

It should be noted that stratigraphic interpretation using CPTU data can also be carried out using a normalized (stress corrected) soil behavior type chart (Robertson, 1990). The Robertson publication emphasizes when normalized stratigraphic interpretation is appropriate.



## Seismic Cone Penetration Testing (SCPTU)

Seismic wave velocity measurements were conducted at regular intervals during the cone penetration test. Seismic wave velocity measurements were made according to the procedures described by Robertson et.al. (1986). Before taking wave velocity measurements, the rods were decoupled from the CPT rig to avoid transmission of energy down the rods.

The seismic waves were generated using a hammer striking a steel beam that was coupled to the ground by a hydraulic cylinder under the CPT rig (refer to Figure S). The sledgehammer striking the beam acts as an electrical contact trigger, initiating the recording of the seismic wave traces. The offset of the beam from the cone was taken into account during calculation of the seismic wave velocities.

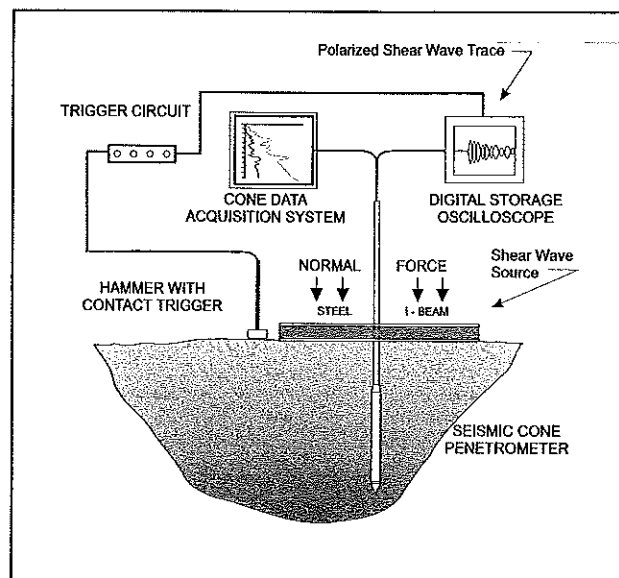


Figure S – Layout of Downhole Seismic Cone System

At each test depth, at least two waves were recorded. Multiple waves are recorded at each end of the beam to enable the operator to check the consistency of the waveforms. The seismic wave receiver used was a horizontally active geophone located in the body of the cone penetrometer. The geophone is located approximately 0.2 meters behind the cone tip. This offset is accounted for in all calculations. Data was sampled at a frequency of 20kHz (i.e. 20,000 samples per second) with a total of 5000 points being recorded per wave trace. To maintain the desired signal resolution, the input sensitivity (gain) of the receiver was increased with depth.

Table SCPTU (Appendix SCPTU) provides a summary of the seismic cone penetration tests carried out. The seismic wave velocity results are presented in both tabular and graphical form in Appendix SCPTU.



## Pore Pressure Dissipation Testing (PPD)

The penetration of the piezocone was halted at specific depths to carry out pore pressure dissipation tests as directed by the field representative. The variation of the penetration pore pressure ( $u$ ) with time was measured and recorded. All pore pressure data was recorded immediately behind the cone tip at the  $u_2$  location (refer to Figure PTL).

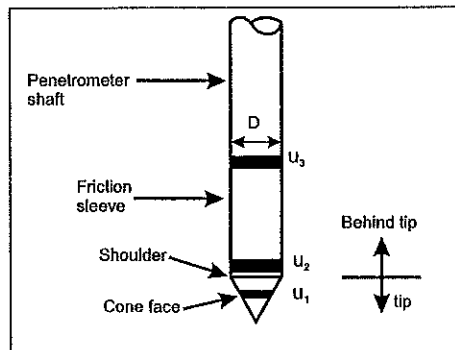


Figure – PTL

Pore pressure dissipation data can be interpreted to provide estimates of :

- equilibrium piezometric pressure
- phreatic surface
- in situ horizontal coefficient of consolidation,  $c_h$
- in situ horizontal coefficient of permeability,  $k_h$

In order to interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until such time as there is no variation in pore pressure with time (refer to Figure PPD). This time is commonly referred to as  $t_{100}$ , the point at which 100% of the excess pore pressure has dissipated.

Interpretation of either  $c_h$  and  $k_h$  from dissipation results can be most easily achieved using either of two analytical approaches: cavity-expansion theory or the strain-path approach. Comparisons of the available solutions and results from field studies suggest that the cavity-expansion method of Torstensson (1977) and the strain-path approaches of Levadous (1980) and Teh (1987) all provide similar predications of consolidation parameters from CPTU dissipation data (Gillespie 1981; Kabir and Lutenegegger 1990; Robertson et al. 1991). Robertson et al. (1991) have shown that these methods, although developed for normally consolidated soils, can be equally applied to overconsolidated soils. Furthermore, comparisons of field and laboratory data indicate that the trends in the measured (laboratory) and predicated (CPTU) data



are consistent provided the micro fabric and nature of the soils being tested are taken into consideration (Danziger 1990; Robertson et al. 1991).

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1991.

The pore pressure dissipation tests are summarized in Table PPD (Appendix PPD). Pore pressure dissipation data is presented in Appendix PPD.

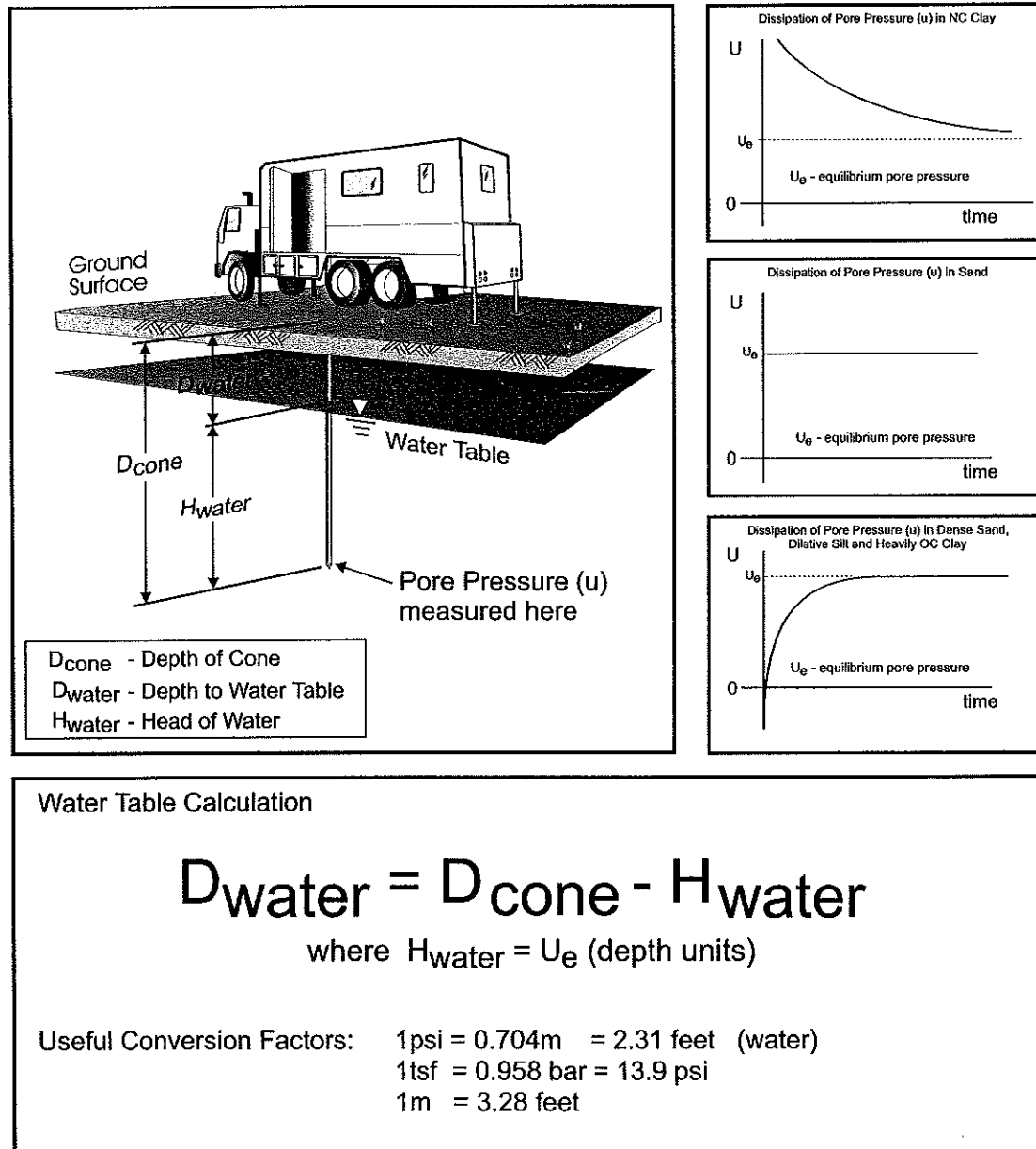


Figure - PPD

## ConeTec Digital File Formats

### CPT Data Files

---

Unless otherwise requested by the client ConeTec CPT data files are named such that the first 3 characters contain the job number, the next two characters are typically CP followed by two characters indicating the sounding number. The last DOS character position is reserved for the letters a, b, c, d etc to uniquely identify multiple soundings at the same location. The CPT sounding file has the extension COR and pore pressure dissipation files have the extension PPD. As an example, for job number 99-127 the first sounding will have file names 127CP01.COR and 127CP01.PPD.

The CPT (COR) file consists of the following components:

1. Two lines of header information
2. Data records
3. End of data marker
4. Units information

#### Header Lines

- Line 1: Columns 1-6 are blank (future use)  
 Columns 7-21 contain the sounding Date and Time  
 Columns 22-36 contain the sounding Operator
- Line 2: Columns 1-16 contain the Job Location  
 Columns 17-31 contain the Cone ID  
 Columns 32-47 contain the sounding number

#### Data Records

The data records contain 4 or more columns of data in floating point format. A comma (and spaces) separates each data item:

- Column 1: Sounding Depth (meters)  
 Column 2: Tip ( $q_c$ ) data uncorrected for pore pressure effects. Recorded in units selected by the operator.  
 Column 3: Sleeve ( $f_s$ ) data. Recorded in units selected by the operator  
 Column 4: Dynamic pore pressure readings. Recorded in units selected by the operator  
 Column 5: Exists only if specialty modules (Resistivity and/or UVIF) have been used

#### End of Data Marker

After the last line of data a line containing ASCII 26 (CTL-Z) and a newline (carriage return/ line feed) character. This is used to mark the end of data.

#### Units Information

The last section of the file contains information about the units that were selected for the sounding. A separator bar makes up the first line. The second line contains the type of units used for depth,  $q_c$ ,  $f_s$  and u. The third line contains the conversion values required for ConeTec's software to convert the recorded data to an internal set of base units (bar for  $q_c$ , bar for  $f_s$  and meters for u).



## CPT Dissipation Files

---

CPT Dissipation files have the same naming convention as the CPT sounding files and have the extension PPD. PPD files consist of the following components:

1. Two lines of header information
2. Data records

### Header Lines (same as COR file):

- Line 1: Columns 1-6 are blank (future use)  
Columns 7-21 contain the sounding Date and Time  
Columns 22-36 contain the sounding Operator
- Line 2: Columns 1-16 contain the Job Location  
Columns 17-31 contain the Cone ID  
Columns 32-47 contain the sounding number

### Data Records

The data records immediately follow the header lines. Each data record can occupy several lines in the file and is a complete record of a dissipation test at a particular depth. Each data record starts with a line containing two values separated by spaces; the first value being an index number (not currently used by the Software) and the second being the dissipation test depth in meters. Following this line are the dissipation pore pressure values stored at 5 second intervals with a maximum of 12 entries per line. The last line of the dissipation record may not contain a full 12 entries. The data record is terminated with an ASCII 30 character (appears as a triangle in some editors).

This sequence is repeated for every dissipation test in the sounding. No marker is used to indicate end of file. Units information is not stored in this file. Users would have to check the CPT file for the units that were used.

## CPT Interpretations

---

ConeTec's CPT interpretation output files are generally delivered in two styles known as IFP and IFI files (printable and importable). One style has page formatting (IFP) and only uses up to 132 columns across the file. This allows the file to be printed on standard office printers. The files are usually formatted for 70 lines per page with form feed characters embedded into the file. They have been designed for use with the HP laserjet lineprinter or compressed (16.7 characters/inch) fixed pitch font style. Multiple pages are required to print an entire sounding. There are multiple parts to each page (pages 1a and 1b) to accommodate all the geotechnical parameters that are output.

The Importable file type (IFI) contains the same data as the IFP files but is set up with all the columns across the file. No page formatting is done. This file is designed for importing into spreadsheet programs.



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Woeller, D.J., P.K. Robertson, T.J. Boyd and Dave Thomas, "Detection of Polyaromatic Hydrocarbon Contaminants Using the UVIF-CPT", 53<sup>rd</sup> Canadian Geotechnical Conference Montreal, QC October pp 733739, 2000.



# Appendix CPTU

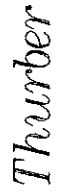
## Cone Penetration Tests



Job No: 04-166  
Client: Thurber Engineering Ltd.  
Project: Highway 69-4 Lane, Estaire, Ontario  
Date: May 15 - 18, 2004

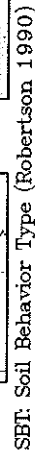
### **CPT SUMMARY**

CPT Sounding	Filename	Date	Type Of Cone	Assumed Water Table (m)	Final Depth (m)
SCPT04-01N	166C01N	05/17/04	20 ton, 15 cm <sup>2</sup>	4.35	46.10
SCPT04-01S	166C01S	05/17/04	20 ton, 15 cm <sup>2</sup>	5.69	49.70
CPT04-02N	166C02N	05/18/04	20 ton, 15 cm <sup>2</sup>	5.60	48.53
CPT04-02S	166C02S	05/18/04	20 ton, 15 cm <sup>2</sup>	4.40	49.68
CPT04-03N	166C03N	05/15/04	20 ton, 15 cm <sup>2</sup>	6.47	46.35
CPT04-03S	166C03S	05/15/04	20 ton, 15 cm <sup>2</sup>	7.30	52.75
CPT04-04N	166C04N	05/18/04	20 ton, 15 cm <sup>2</sup>	5.85	28.65
CPT04-04S	166C04S	05/16/04	20 ton, 15 cm <sup>2</sup>	3.95	37.93



Location: HWY69 4L ESTAIRS

0804070604



### Estimated Phreatic Surface

Max. Depth: 46.10 (m)

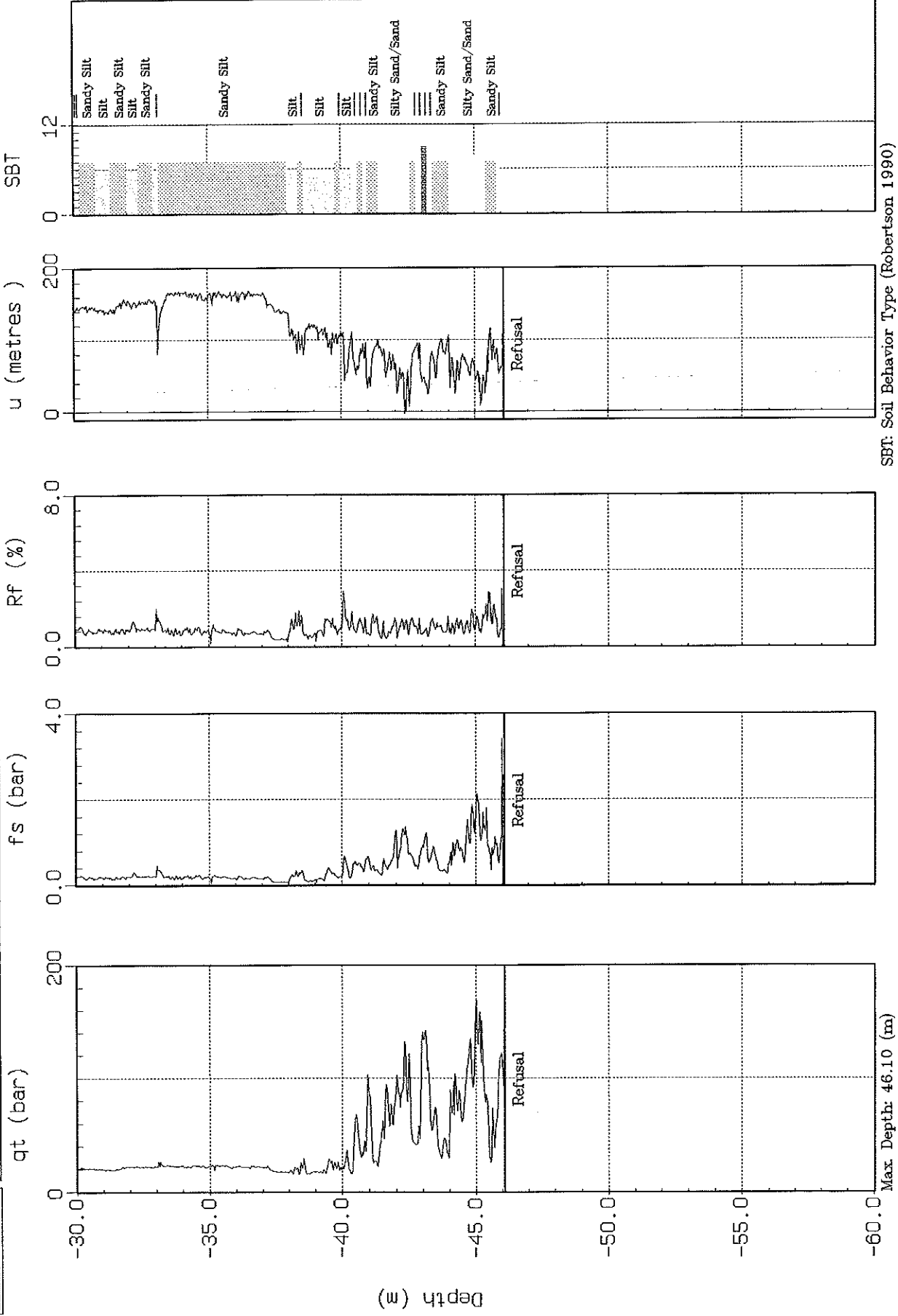
Depth Inc.: 0.025 (m)



Thurber

Site: SCPT04-01N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:17:04 08:09



SBT: Soil Behavior Type (Robertson 1990)

Max Depth: 46.10 (m)

Depth Inc.: 0.025 (m)





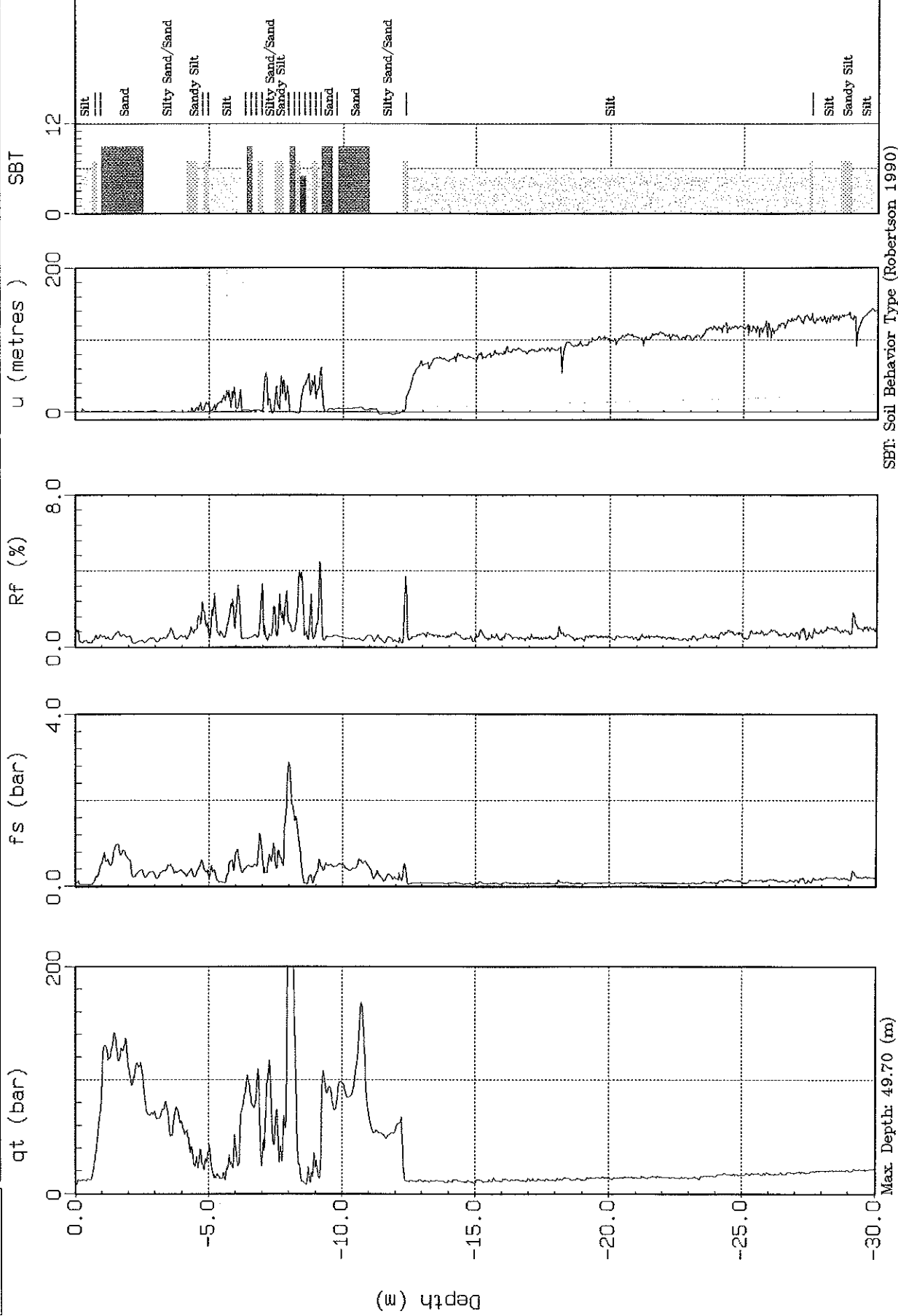
Thurber

Site: SCPT04-01S

Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113

Date: 05:17:04 12:50



SBT: Soil Behavior Type (Robertson 1990)

Estimated Phreatic Surface

Max Depth: 49.70 (m)

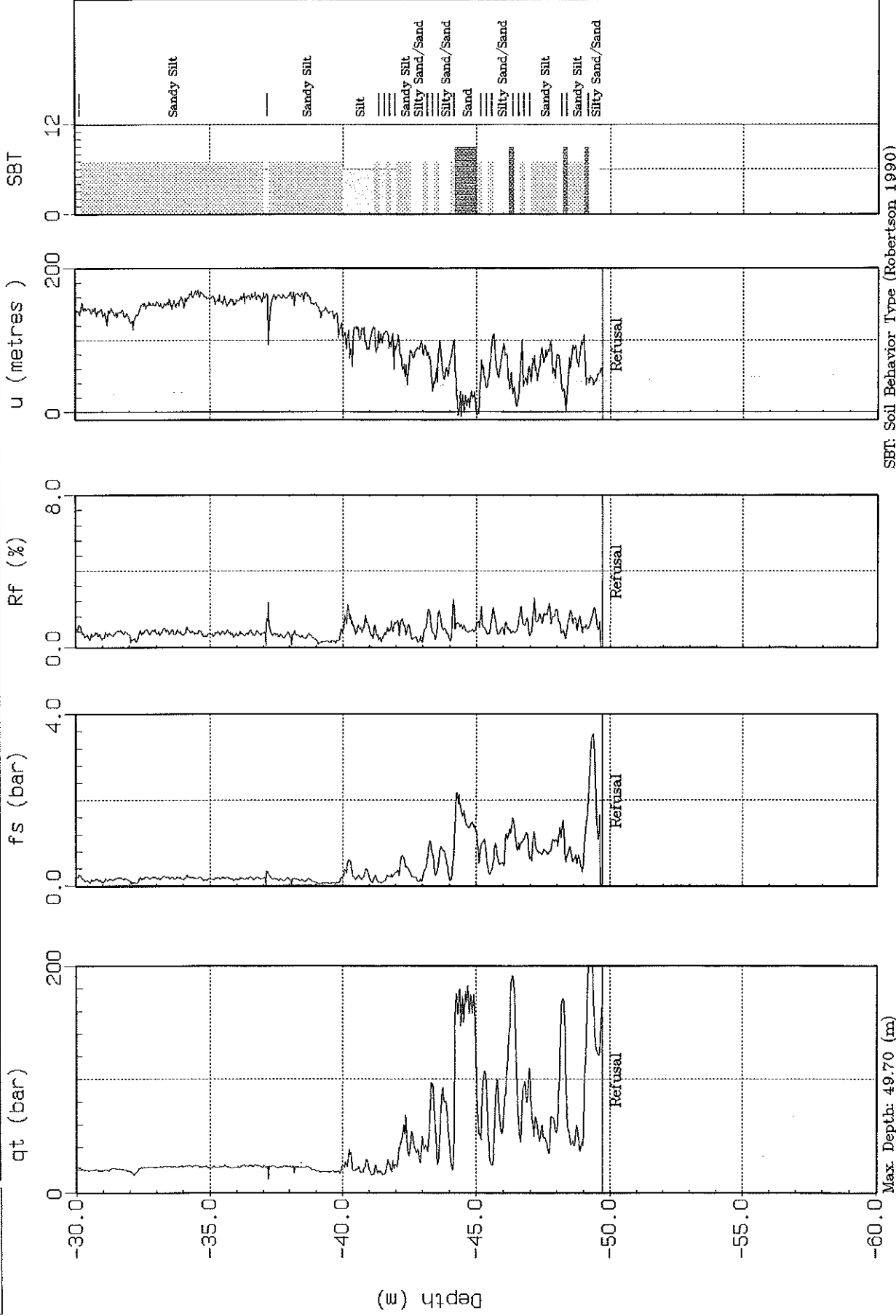
Depth Inc.: 0.025 (m)



Thurber

Site: SCPT04-01S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:17:04 12:50



SBT: Soil Behavior Type (Robertson 1990)

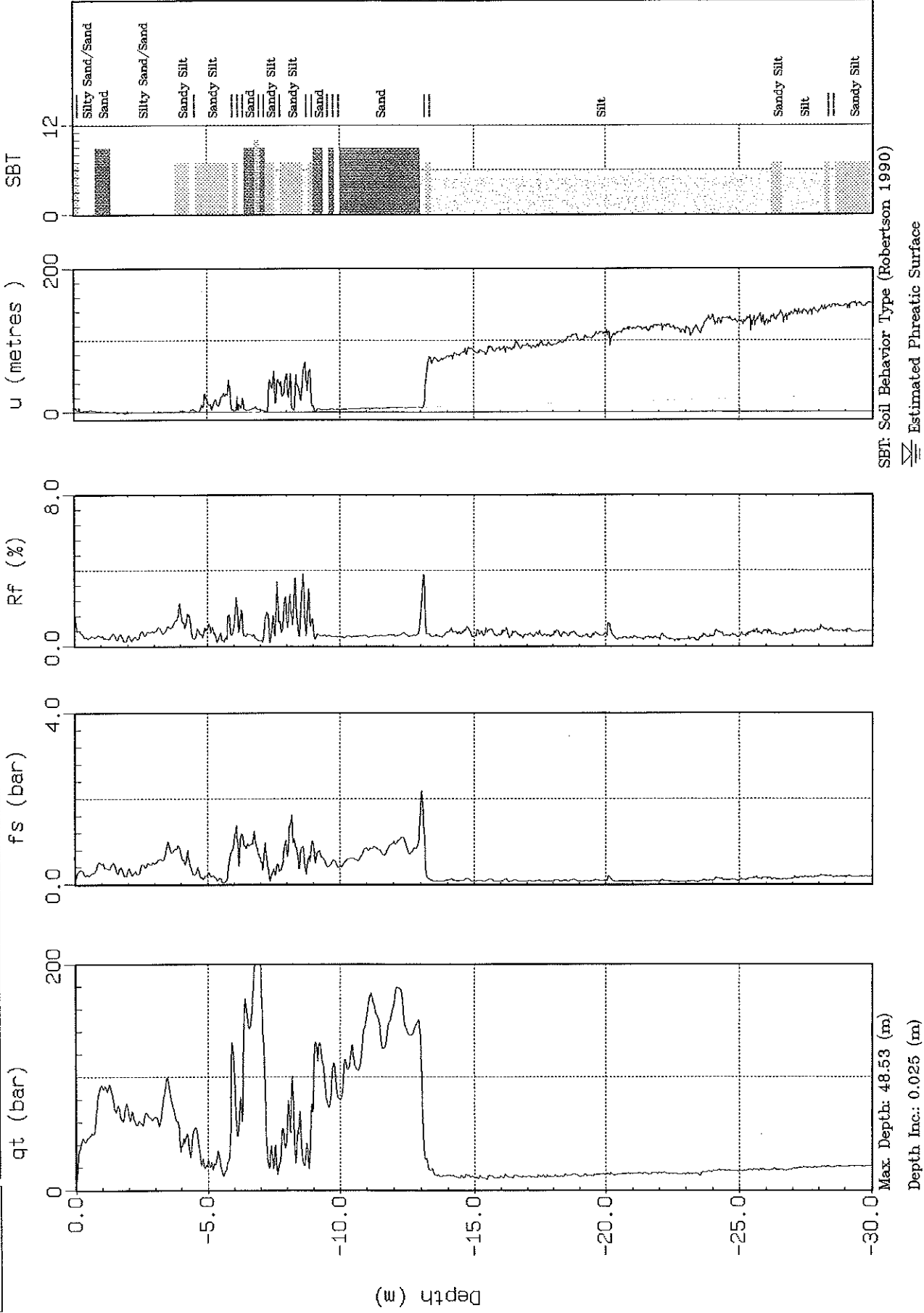
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Depth Inc: 0.025 (m)



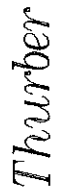
Thurber

Site: CPT04-02N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:18:04 11:22

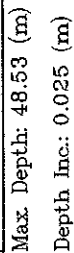


SBT: Soil Behavior Type (Robertson 1990)  
Estimated Phreatic Surface



Location: HWY 69 4L ESTAIRE

Date: 05:18:04 11:22



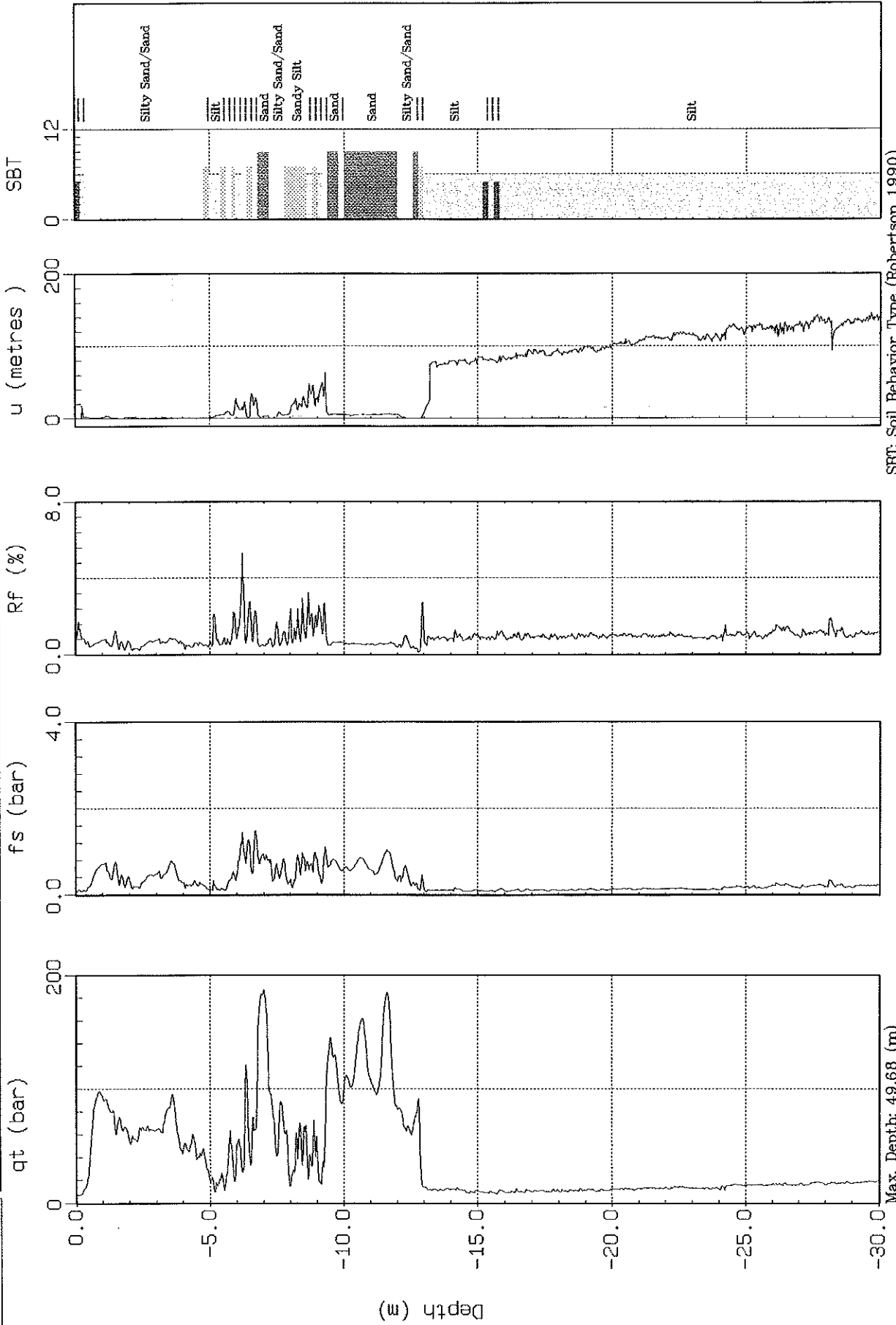
SBT: Soil Behavior Type (Robertson 1990)



Thurber

Site: CPT04-02S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:18:04 08:09

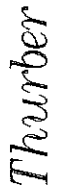


SBT: Soil Behavior Type (Robertson 1990)

Estimated Phreatic Surface

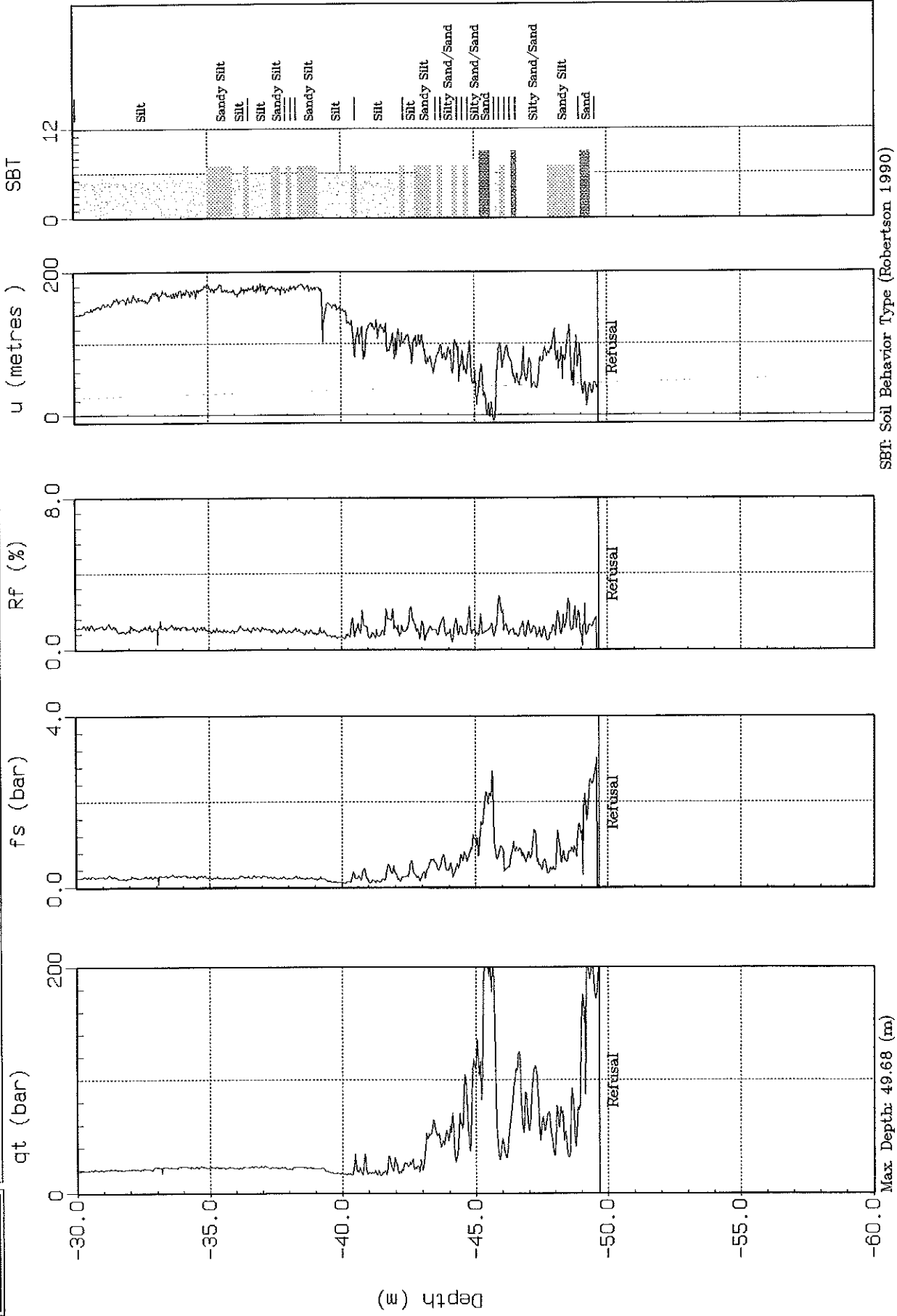
Max. Depth: 49.68 (m)

Depth Inc.: 0.025 (m)



WHI

Cone: 20 Ton St 113  
Date: 05:18:04 08:09



SBT: Soil Behavior Type (Robertson 1990)

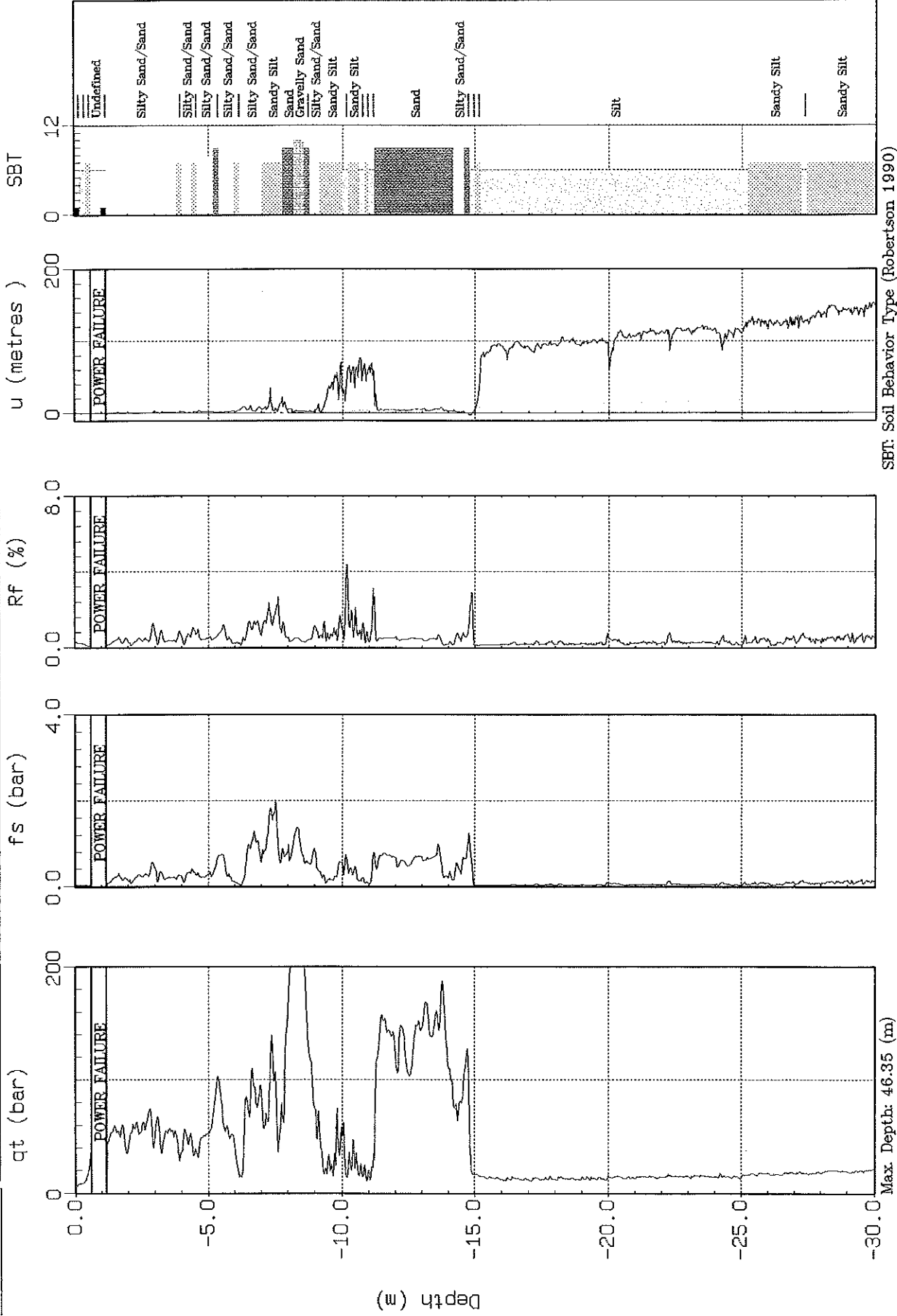
Max Depth: 49.68 (m)  
Depth Inc.: 0.025 (m)



Thurber

Site: CPT04-03N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:15:04 13:06



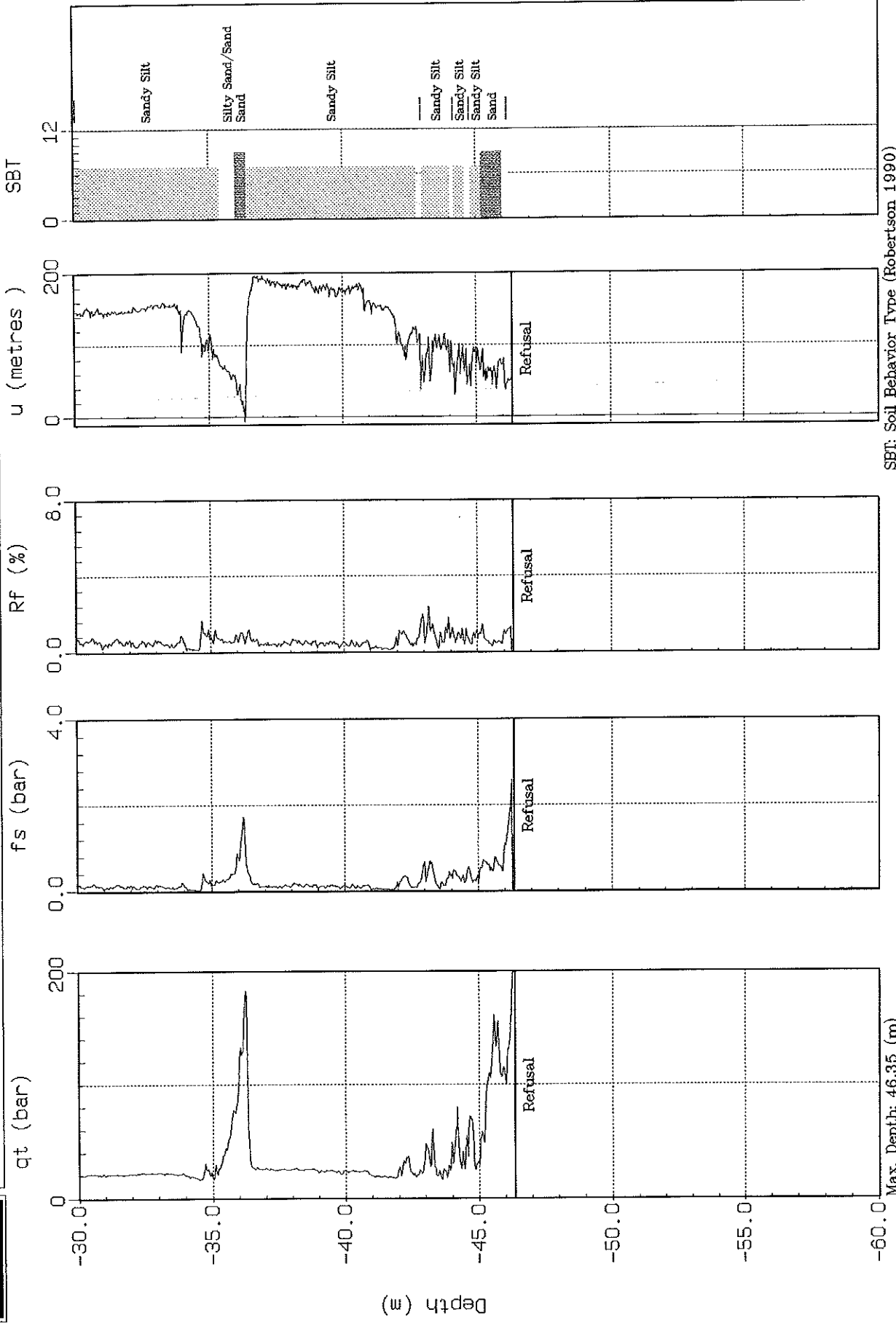
SBT: Soil Behavior Type (Robertson 1990)  
Estimated Phreatic Surface



Thurber

Site: CPT04-03N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:15:04 13:06



SBT: Soil Behavior Type (Robertson 1990)

Max. Depth: 46.35 (m)  
Depth Inc.: 0.025 (m)

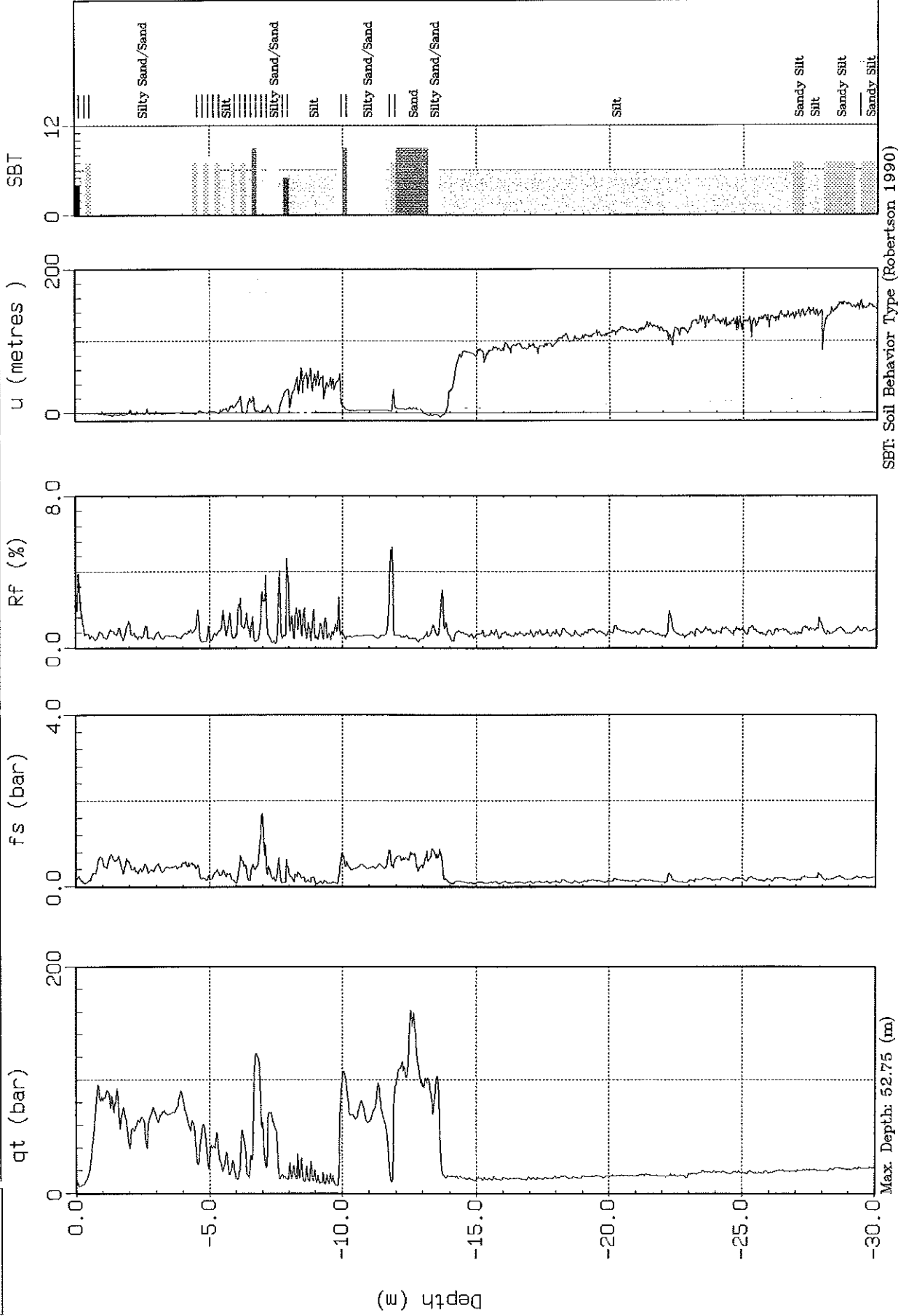




Thurber

Site: CPT04-03S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:15:04 07:59



SBT: Soil Behavior Type (Robertson 1990)

Estimated Phreatic Surface

Max Depth: 52.75 (m)

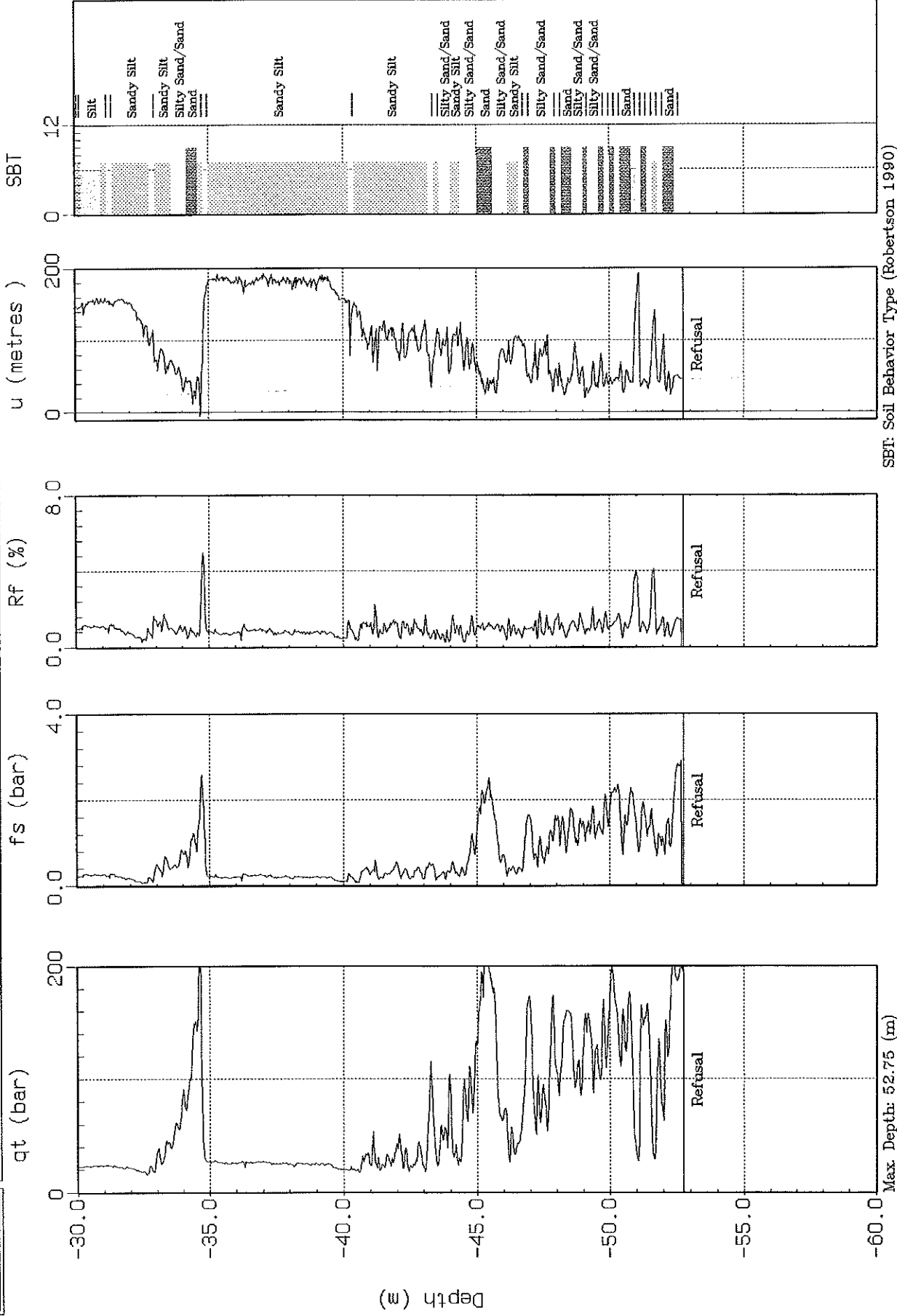
Depth Inc.: 0.025 (m)



Thurber

Site: CPT04-03S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:15:04 07:59



SBT: Soil Behavior Type (Robertson 1990)

Max. Depth: 52.75 (m)  
Depth Inc.: 0.025 (m)



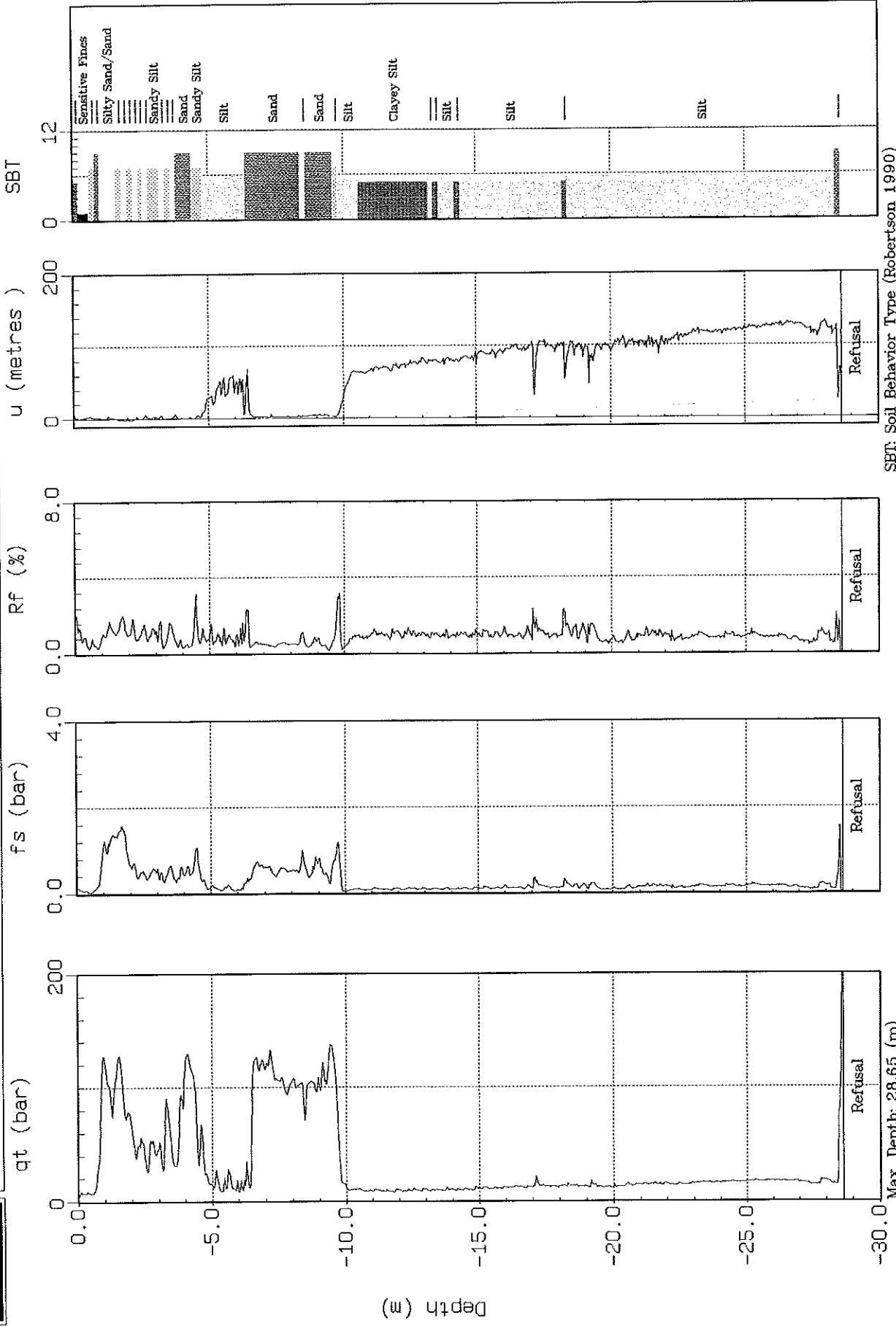
Thurber

Site: CPT04-04N

Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113

Date: 05:18:04 14:36



SBT: Soil Behavior Type (Robertson 1990)

Estimated Phreatic Surface

Max. Depth: 28.65 (m)

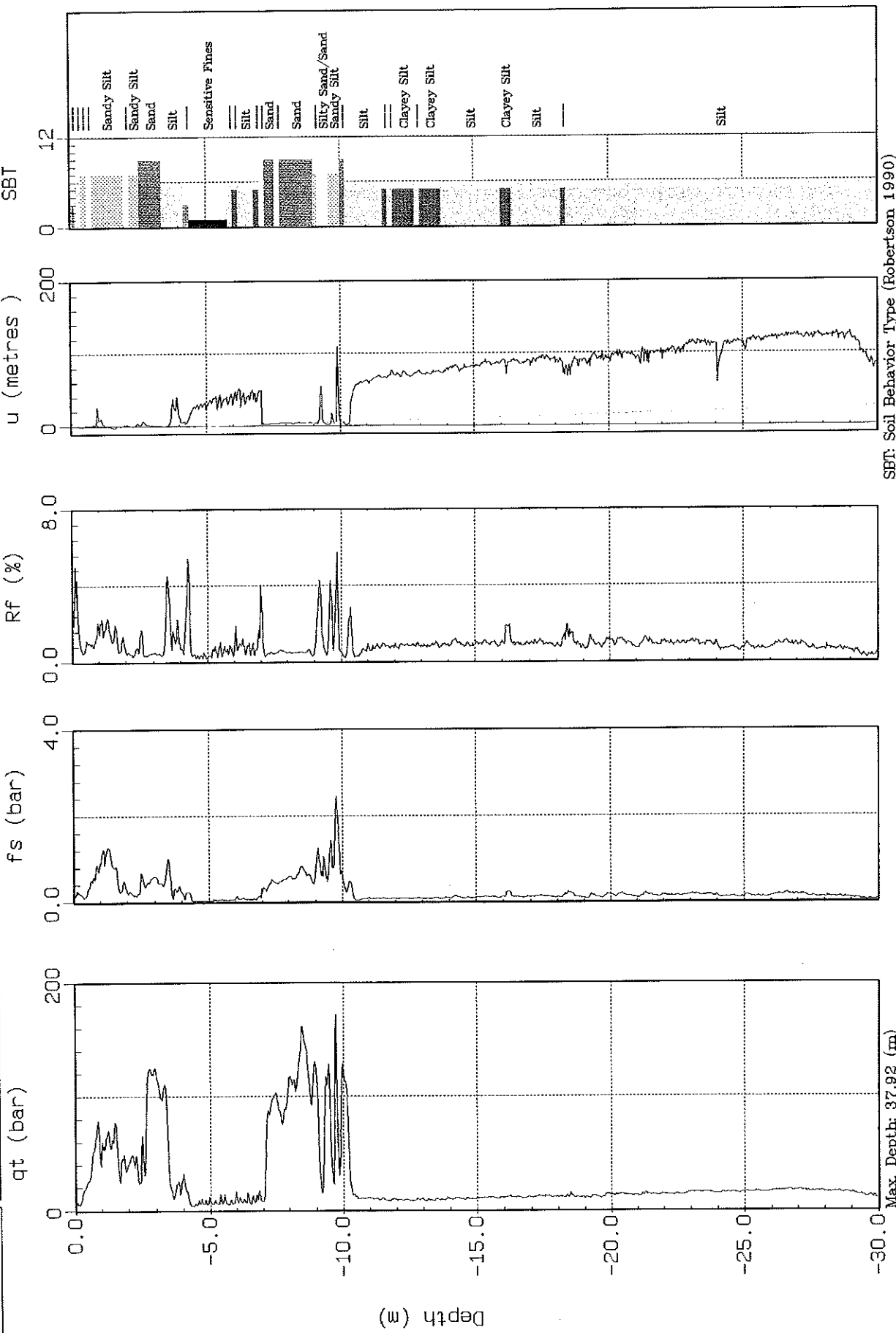
Depth Inc.: 0.025 (m)



Thurber

Site: CPT04-04S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:16:04 07:25



SBT: Soil Behavior Type (Robertson 1990)  
Estimated Phreatic Surface

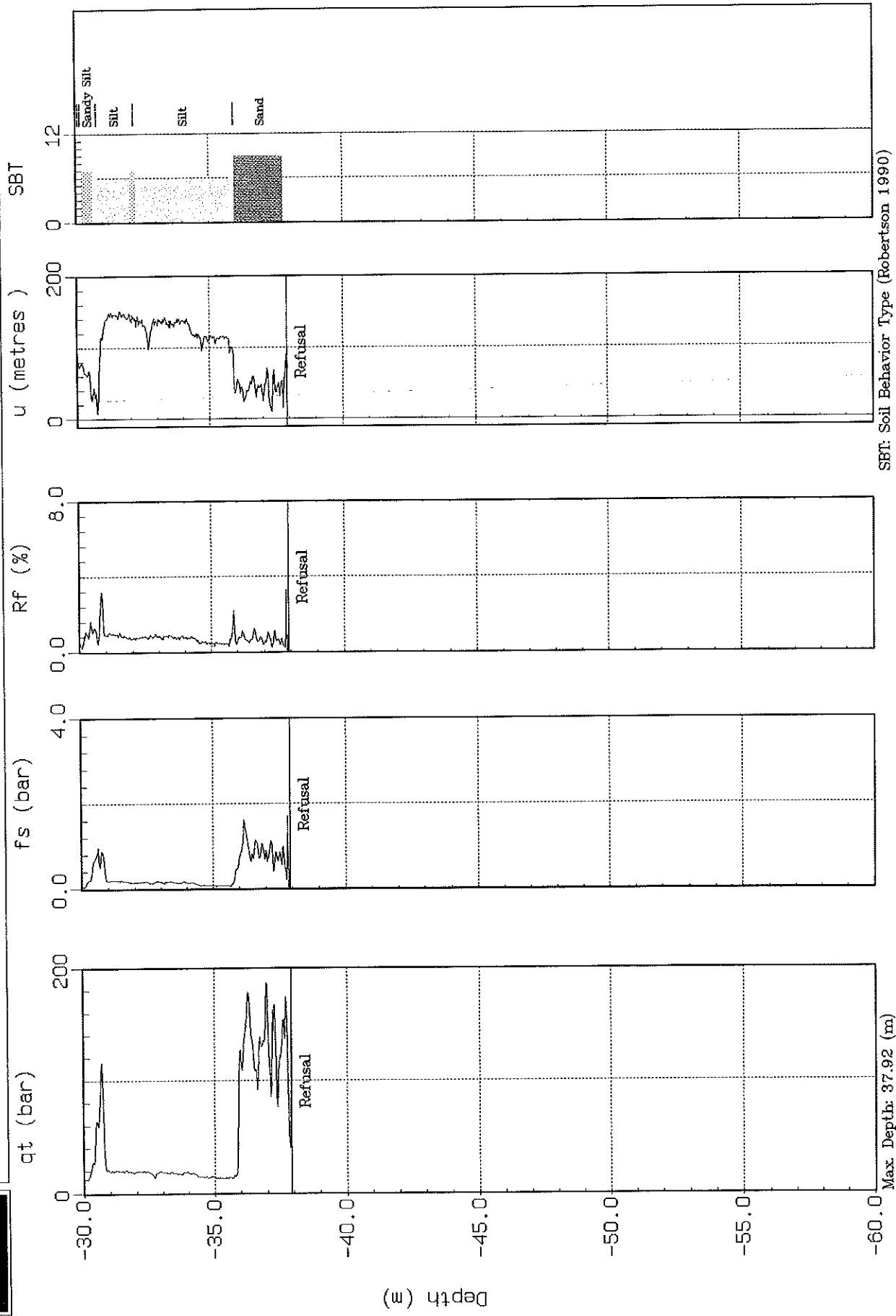
Max Depth: 37.92 (m)  
Depth Inc: 0.025 (m)



Thurber

Site: CPT04-04S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:16:04 07:25

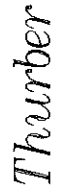


SBT: Soil Behavior Type (Robertson 1990)

Max Depth: 37.92 (m)  
Depth Inc.: 0.025 (m)

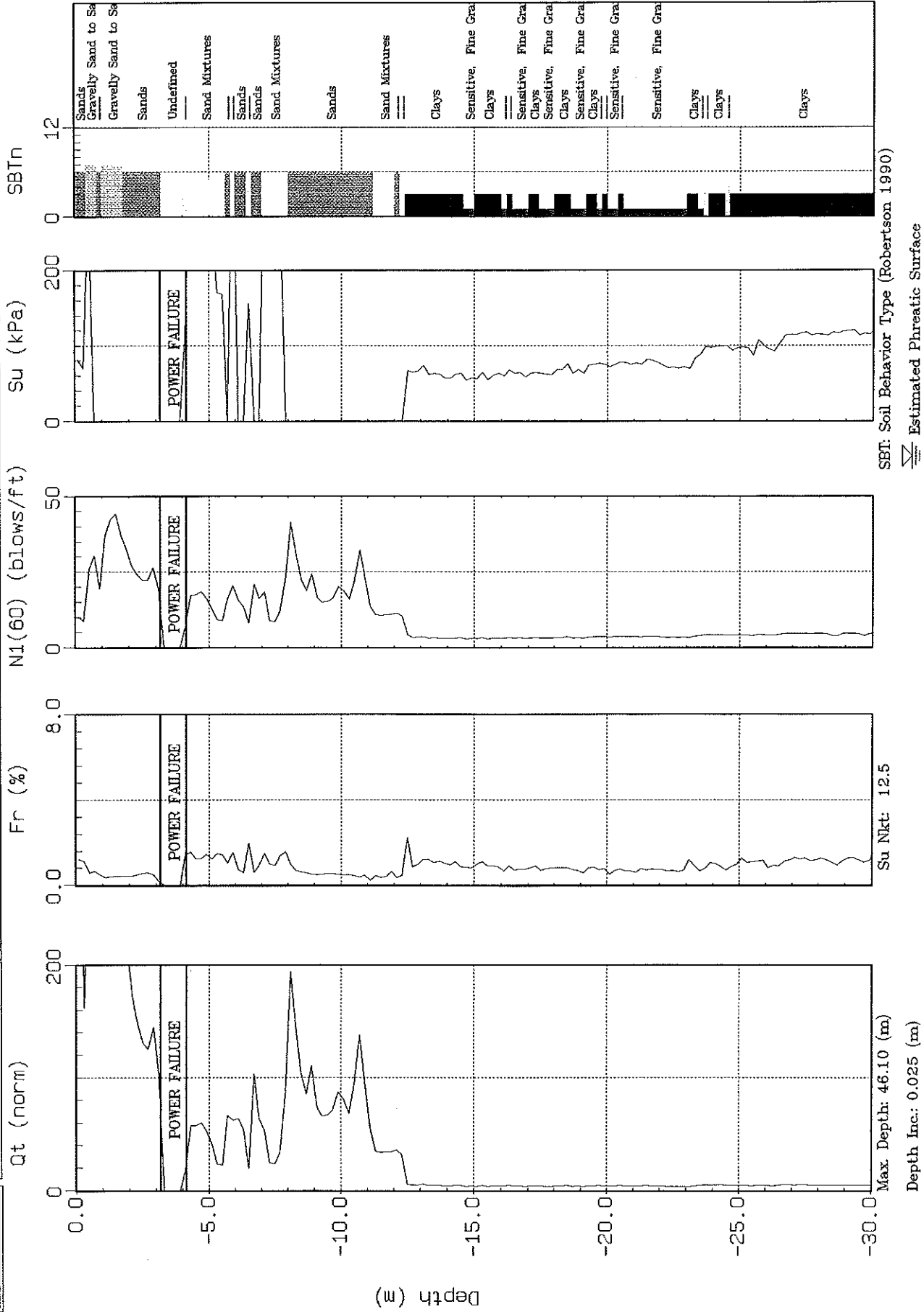
# Appendix CPTU

## Advanced Cone Penetration Tests



Cone: 20 Ton St 113

DATE 17-04-88

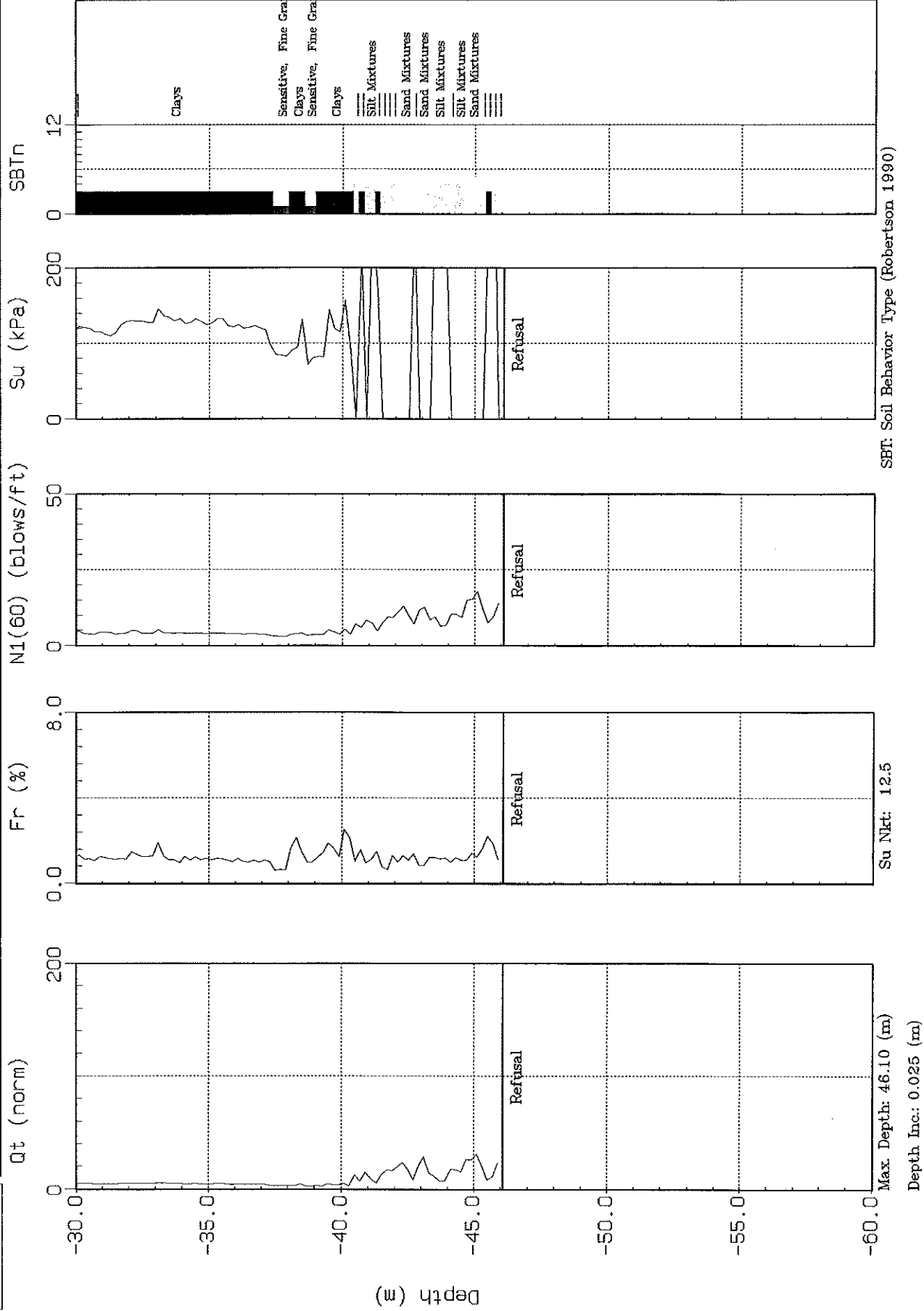




Thurber

Site: SCPT04-01N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:17:04 08:09







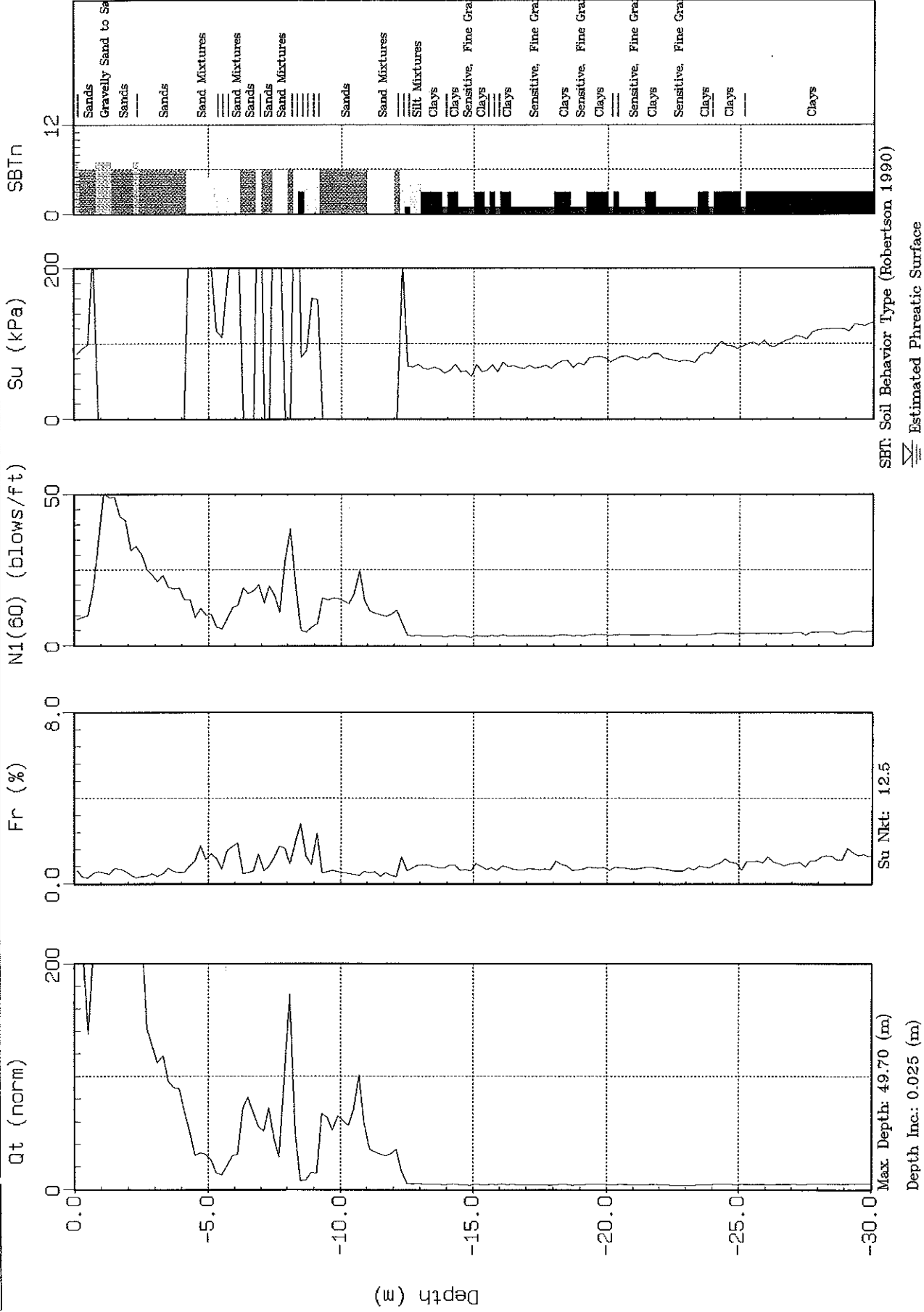
Thurber

Site: SCPT04-01S

Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113

Date: 05:17:04 12:50





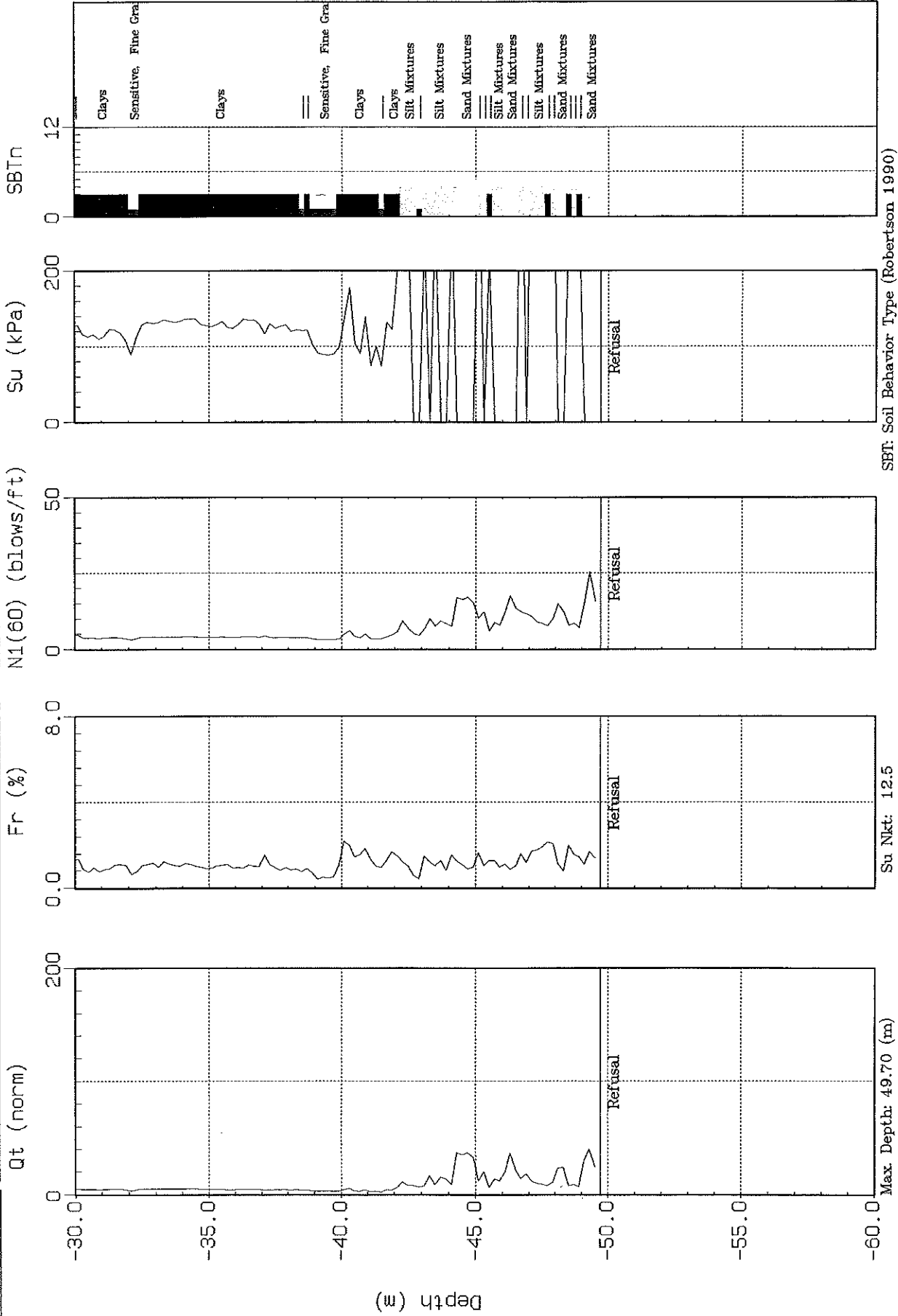
Thurber

Site: SCPT04-01S

Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113

Date: 05:17:04 12:50



SBT: Soil Behavior Type (Robertson 1990)

Su Nkt: 12.5

Max Depth: 49.70 (m)

Depth Inc.: 0.025 (m)



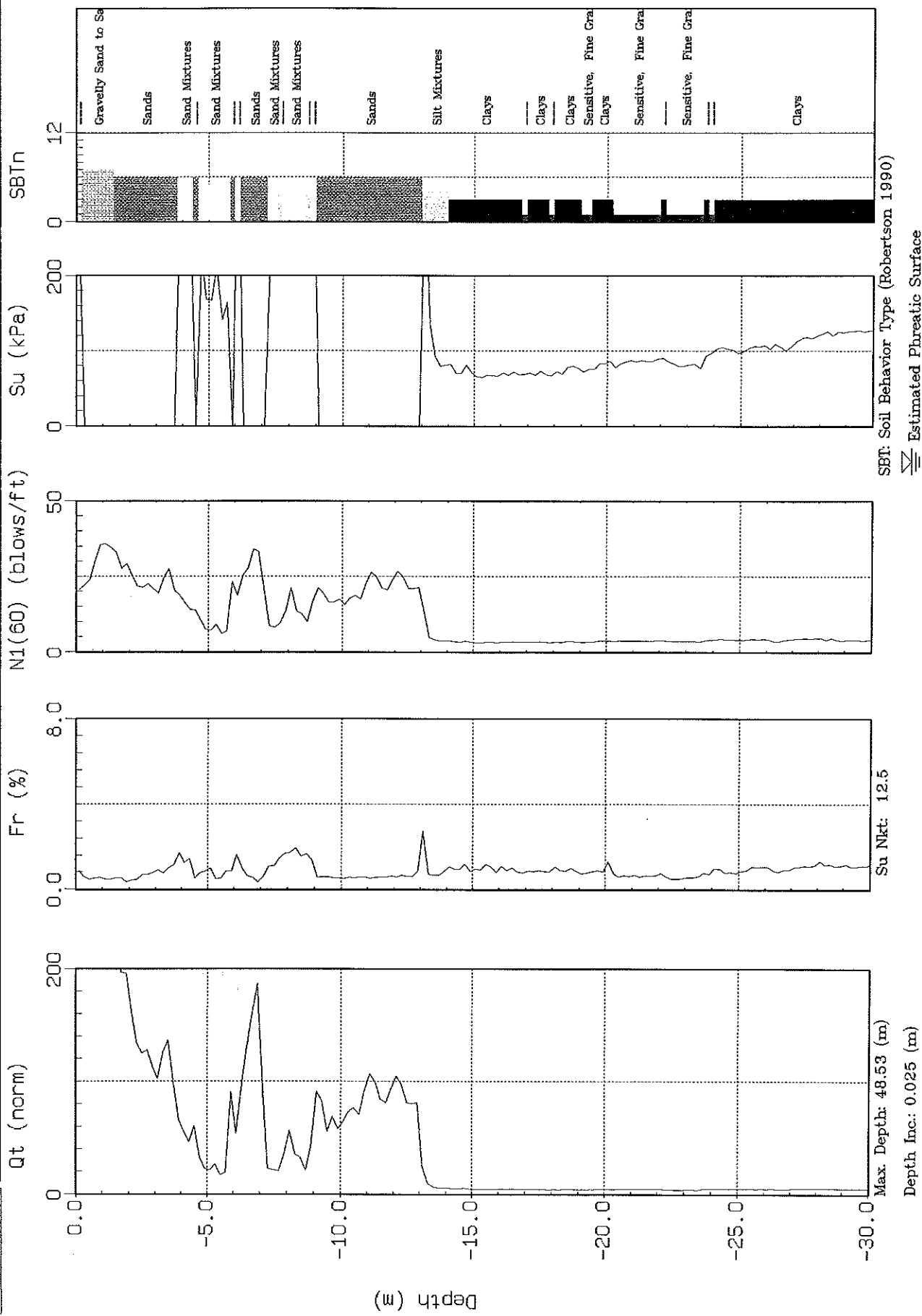
Thurber

Site: CPT04-02N

Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113

Date: 05:18:04 11:22





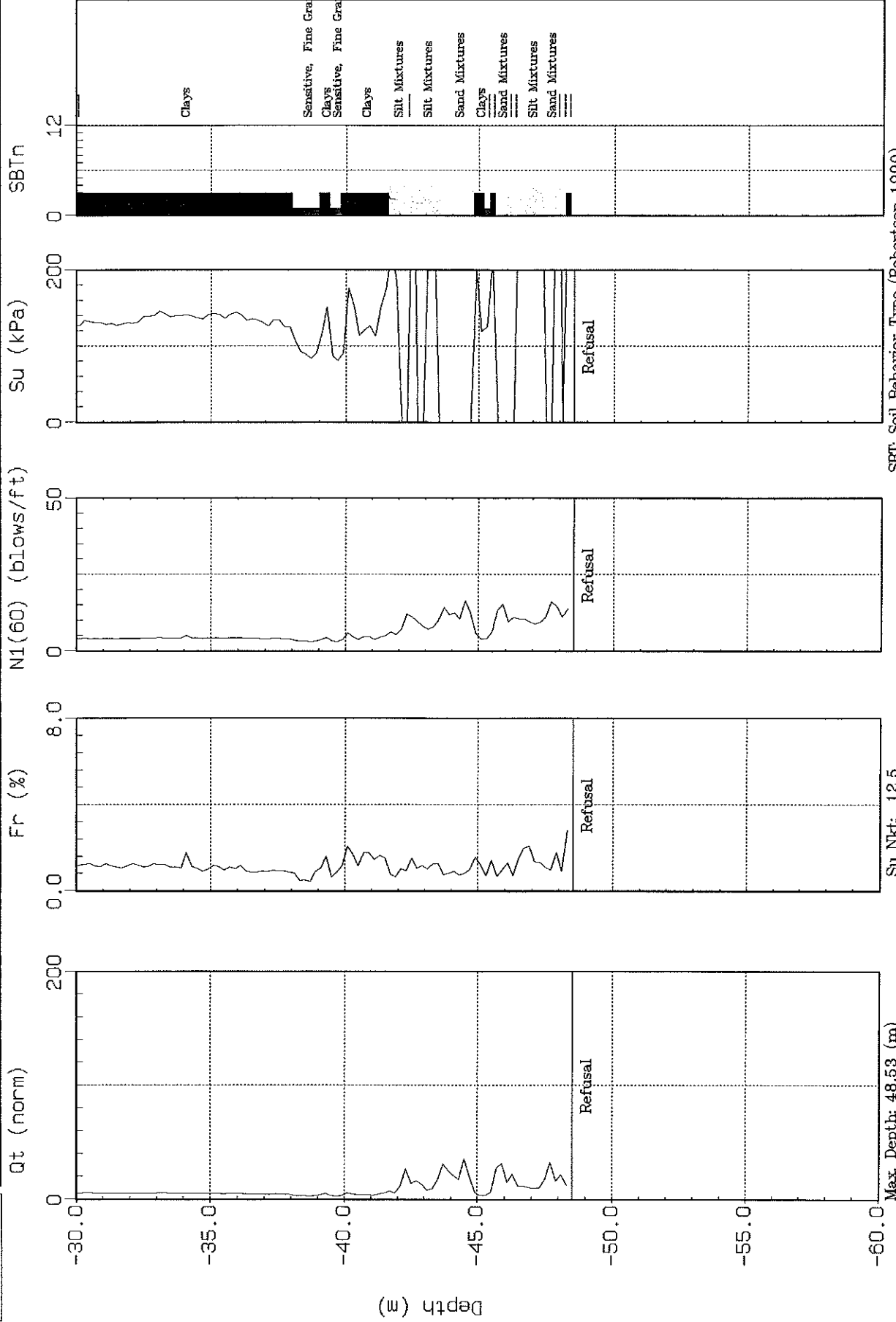
Thurber

Site: CPT04-02N

Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113

Date: 05:18:04 11:22



SBT: Soil Behavior Type (Robertson 1990)

Su Nkt: 12.5

Max. Depth: 48.53 (m)

Depth Inc.: 0.025 (m)



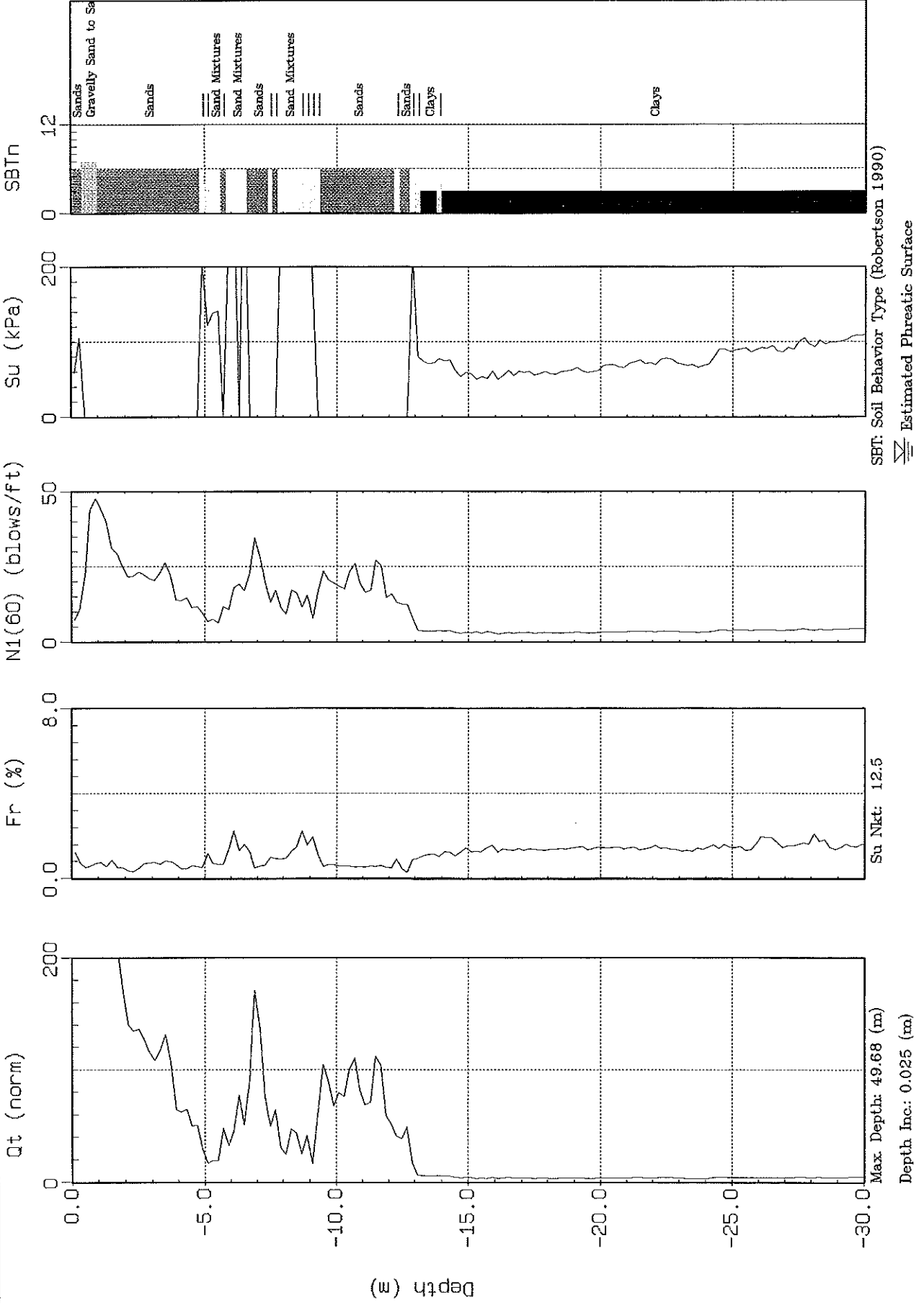
Thurber

Site: CPT04-02S

Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113

Date: 05:18:04 08:09

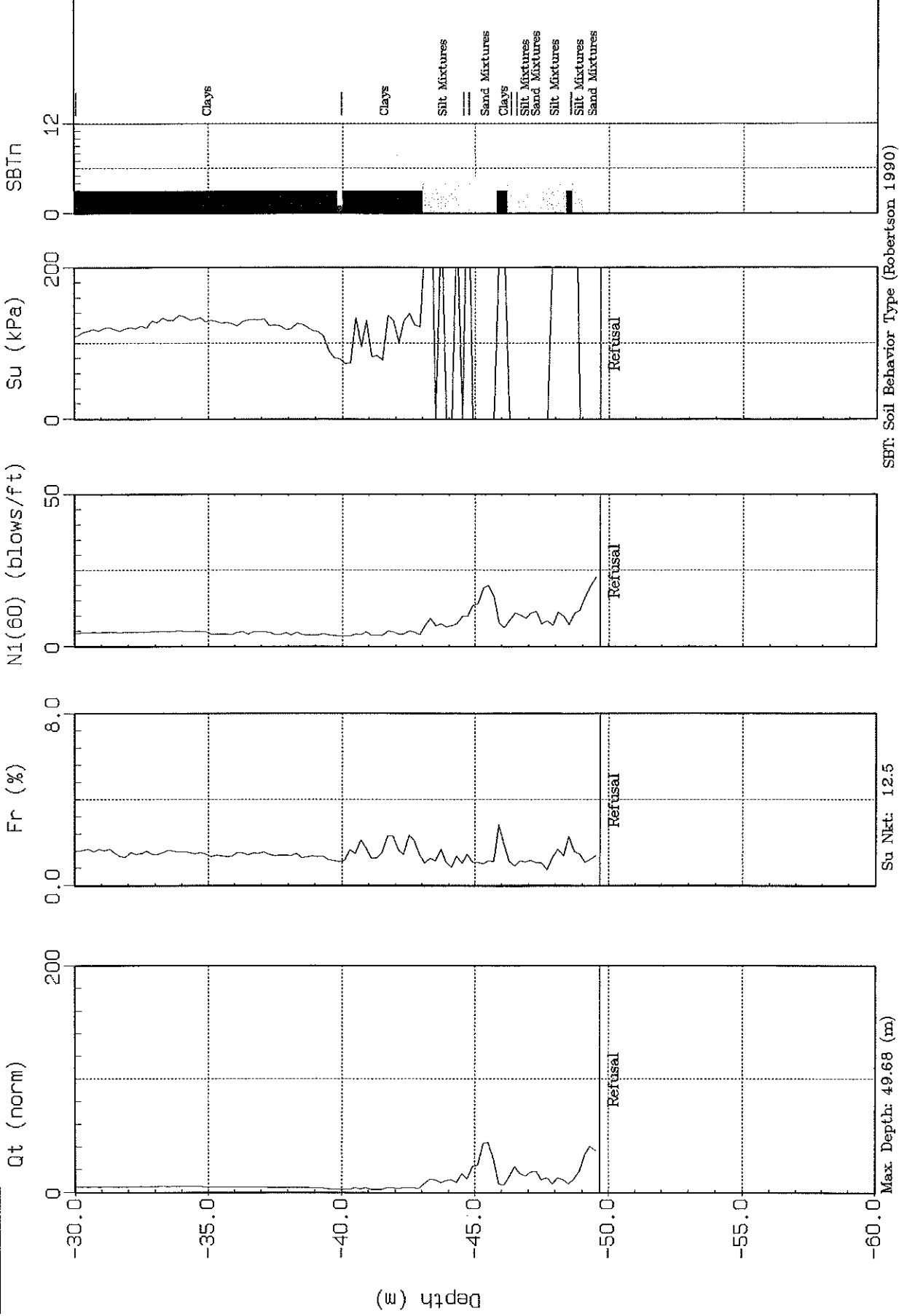




Thurber

Site: CPT04-02S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:18:04 08:09



SBT: Soil Behavior Type (Robertson 1990)

Su Nkt: 12.5

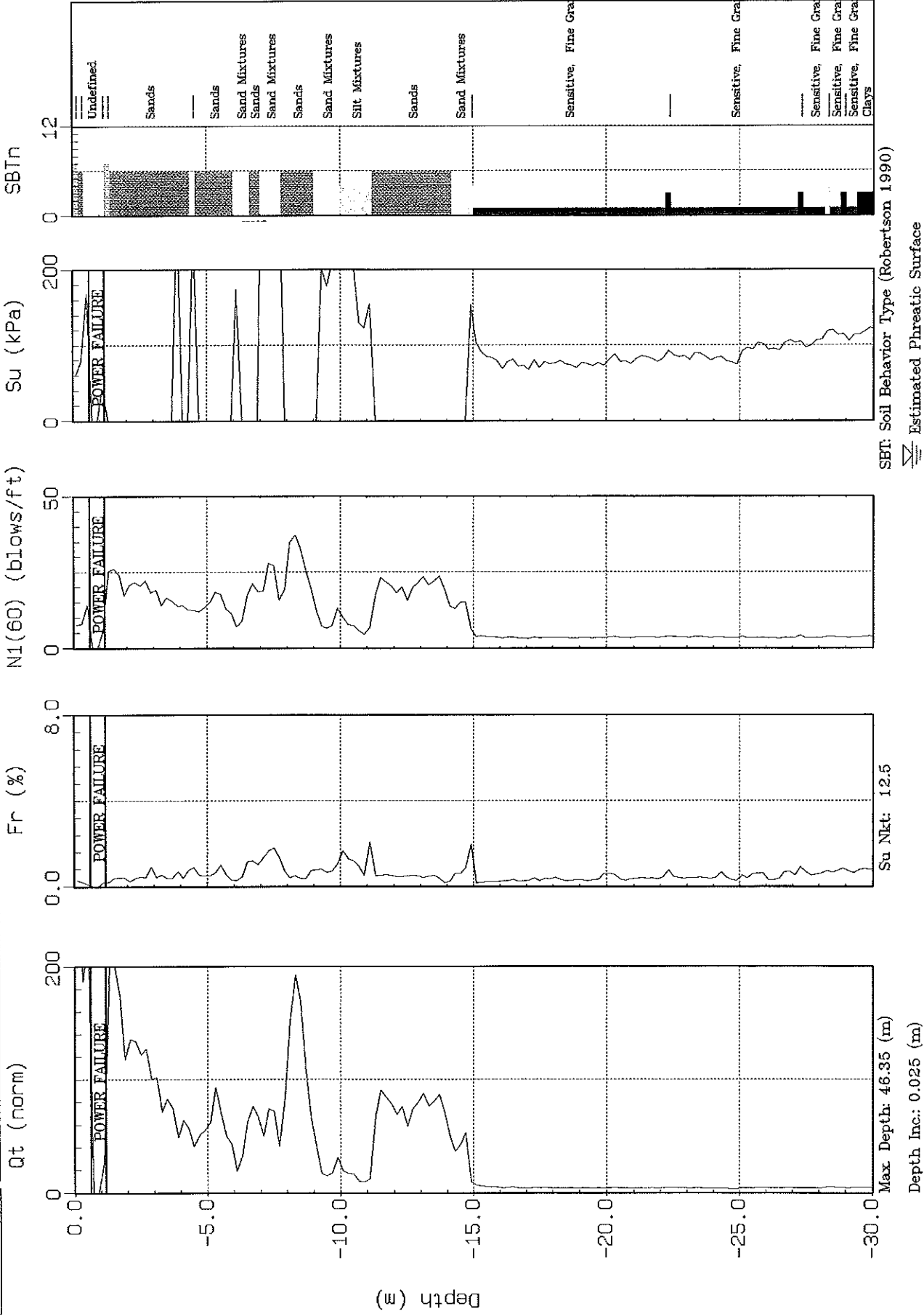
Max Depth: 49.68 (m)  
Depth Inc.: 0.025 (m)



Thurber

Site: CPT04-03N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:15:04 13:06

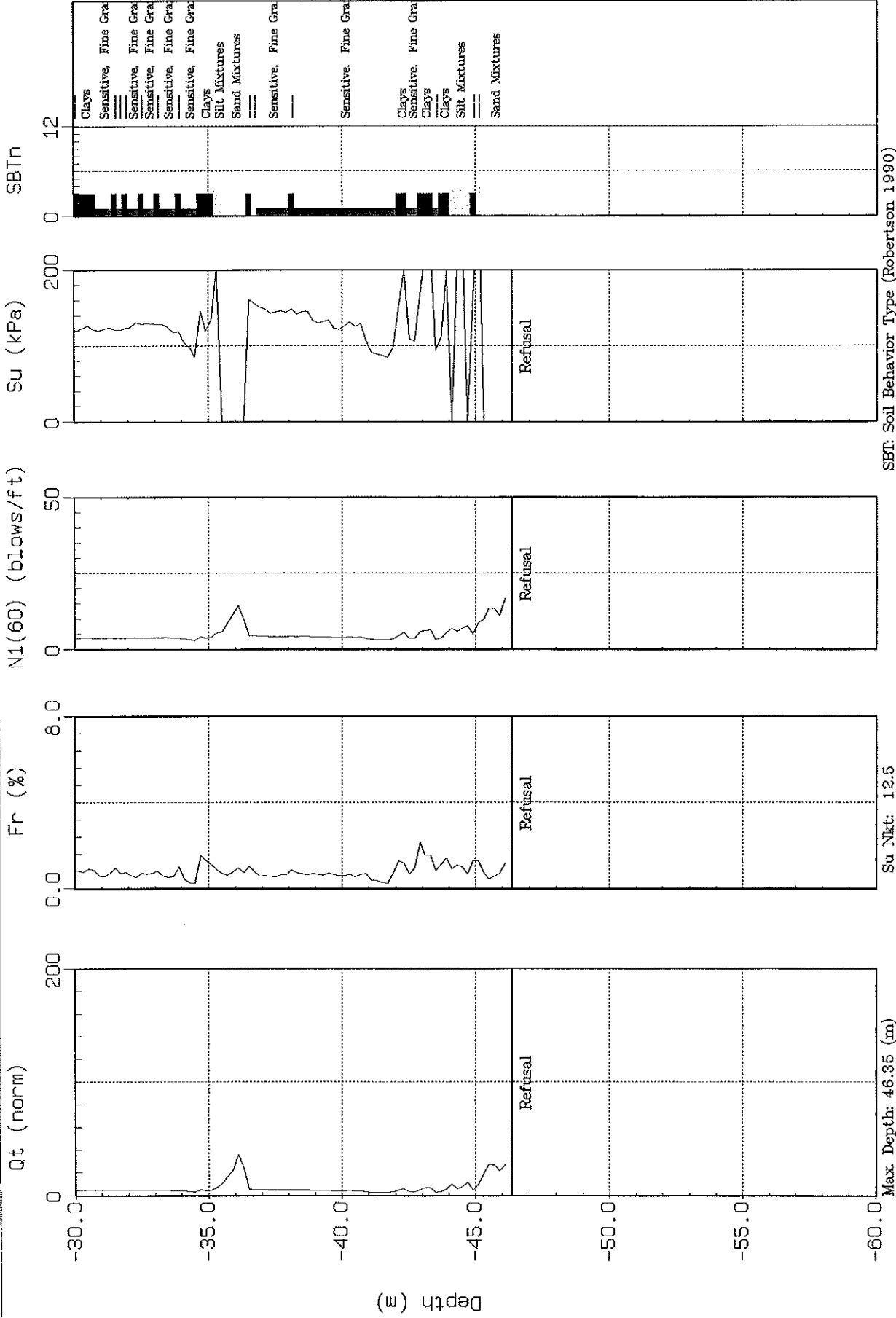




Thurber

Site: CPT04-03N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:15:04 13:06



Su Nkt: 12.5

Max Depth: 46.35 (m)

Depth Inc.: 0.025 (m)

SBT: Soil Behavior Type (Robertson 1990)





Thurber

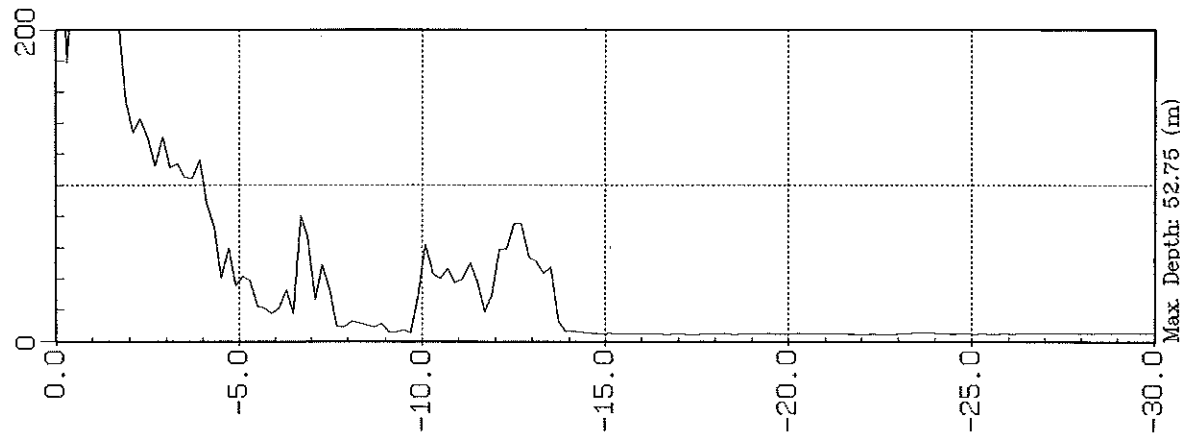
Site: CPT04-03S

Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113

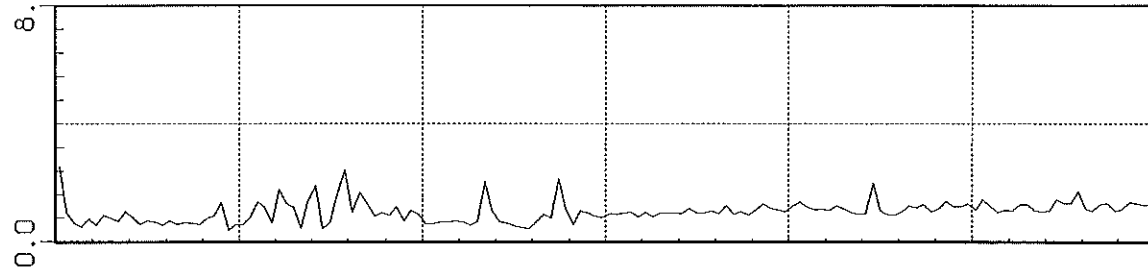
Date: 05:15:04 07:59

Qt (norm)



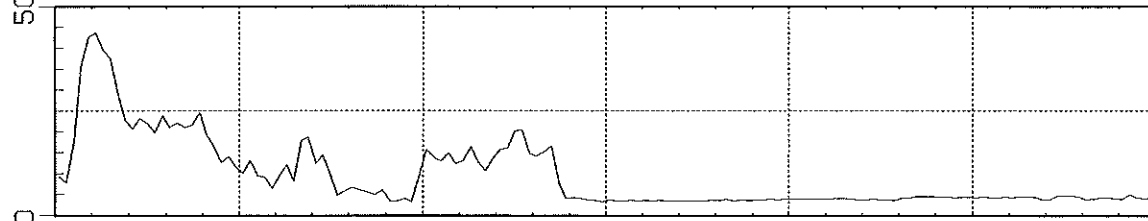
Max Depth: 52.75 (m)  
Depth Inc.: 0.025 (m)

Fr (%)

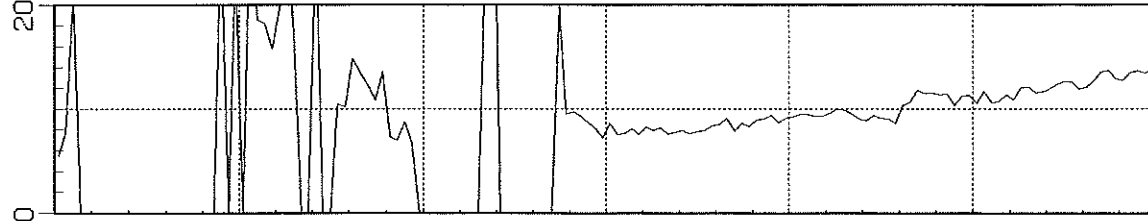


Su Nkt: 12.5

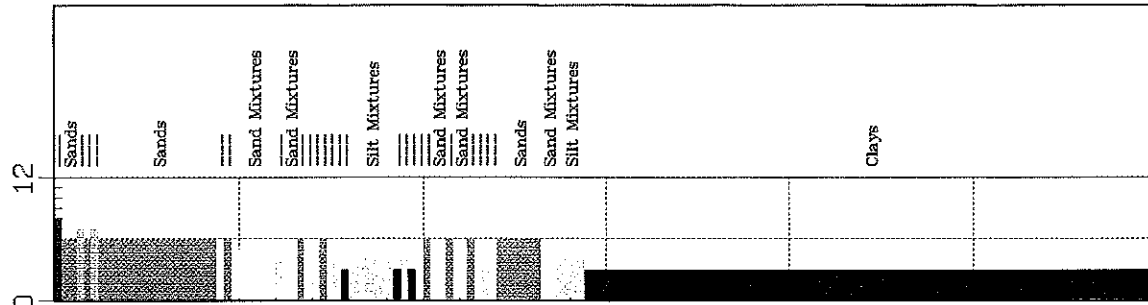
N1(60) (blows/ft)



Su (kPa)



SBIn



SBT: Soil Behavior Type (Robertson 1990)

Estimated Phreatic Surface



Thurber

Site: CPT04-03S

Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113

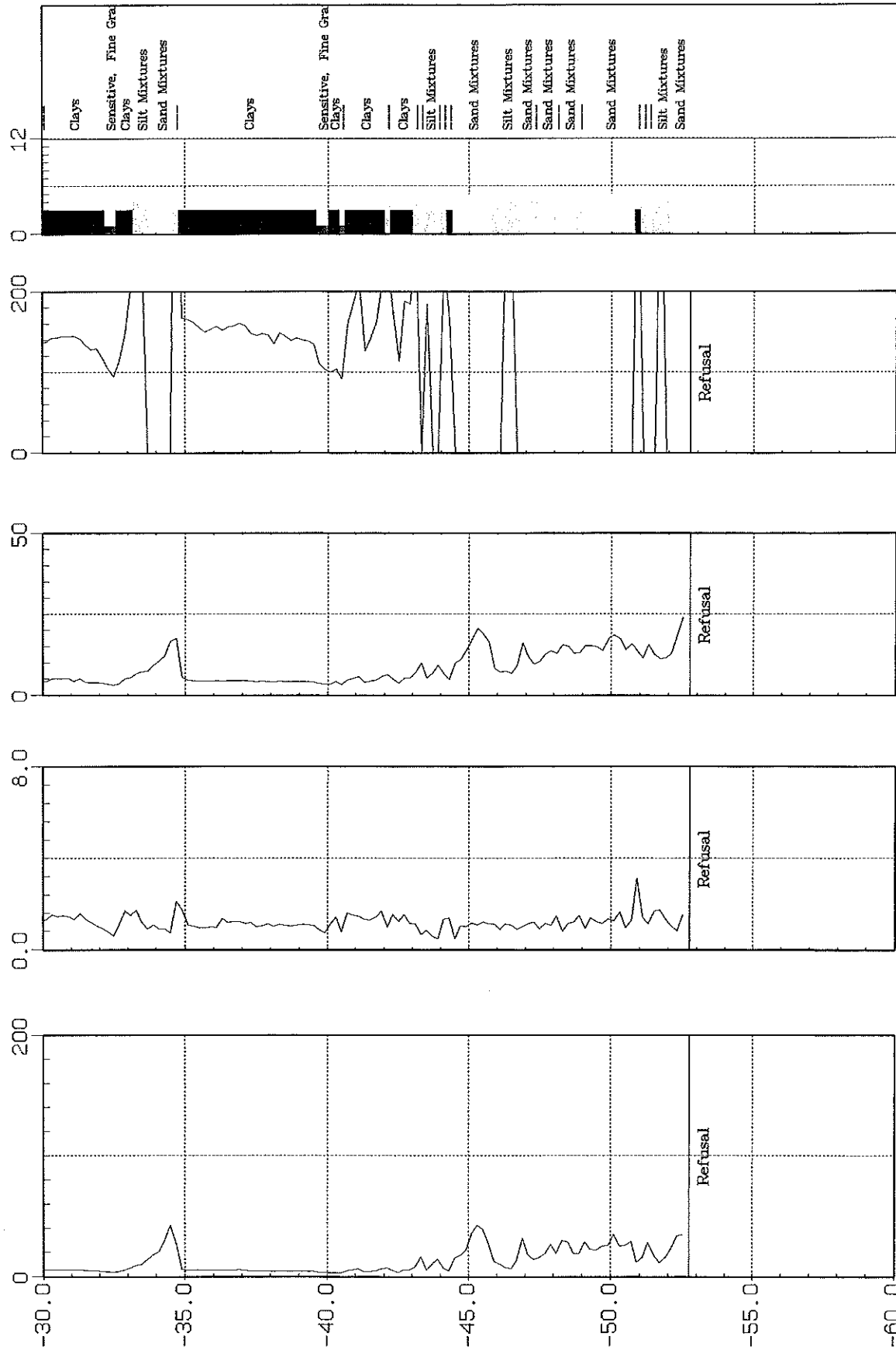
Date: 05:15:04 07:59

Qt (norm)

N1(60) (blows/ft)

Su (kPa)

SBIn

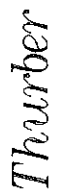


SBT: Soil Behavior Type (Robertson 1990)

Su Nkt: 12.5

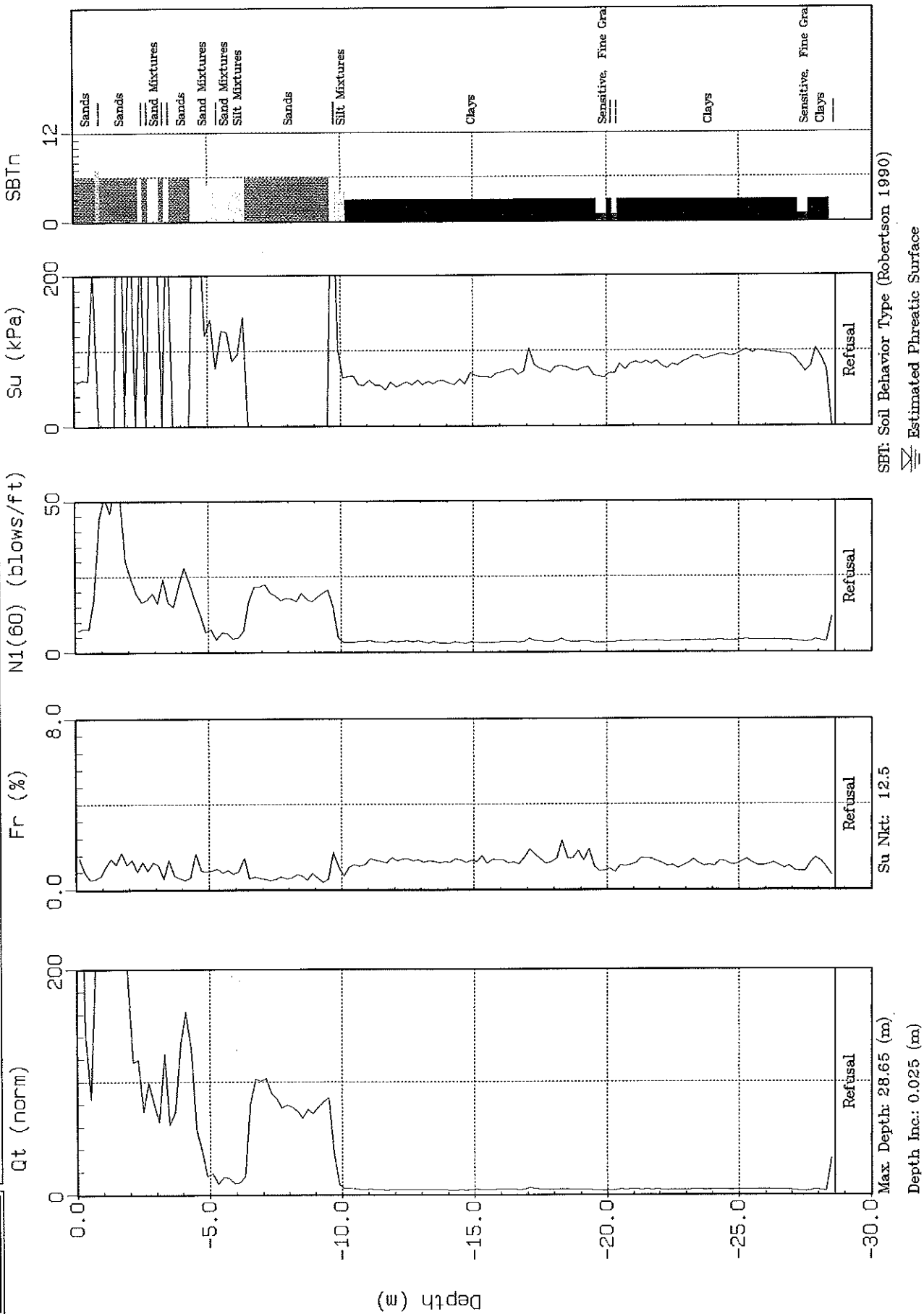
Max Depth: 52.75 (m)

Depth Inc.: 0.025 (m)



Location: HWY69 4L ESTAIRE

Date: 05:18:04 14:36





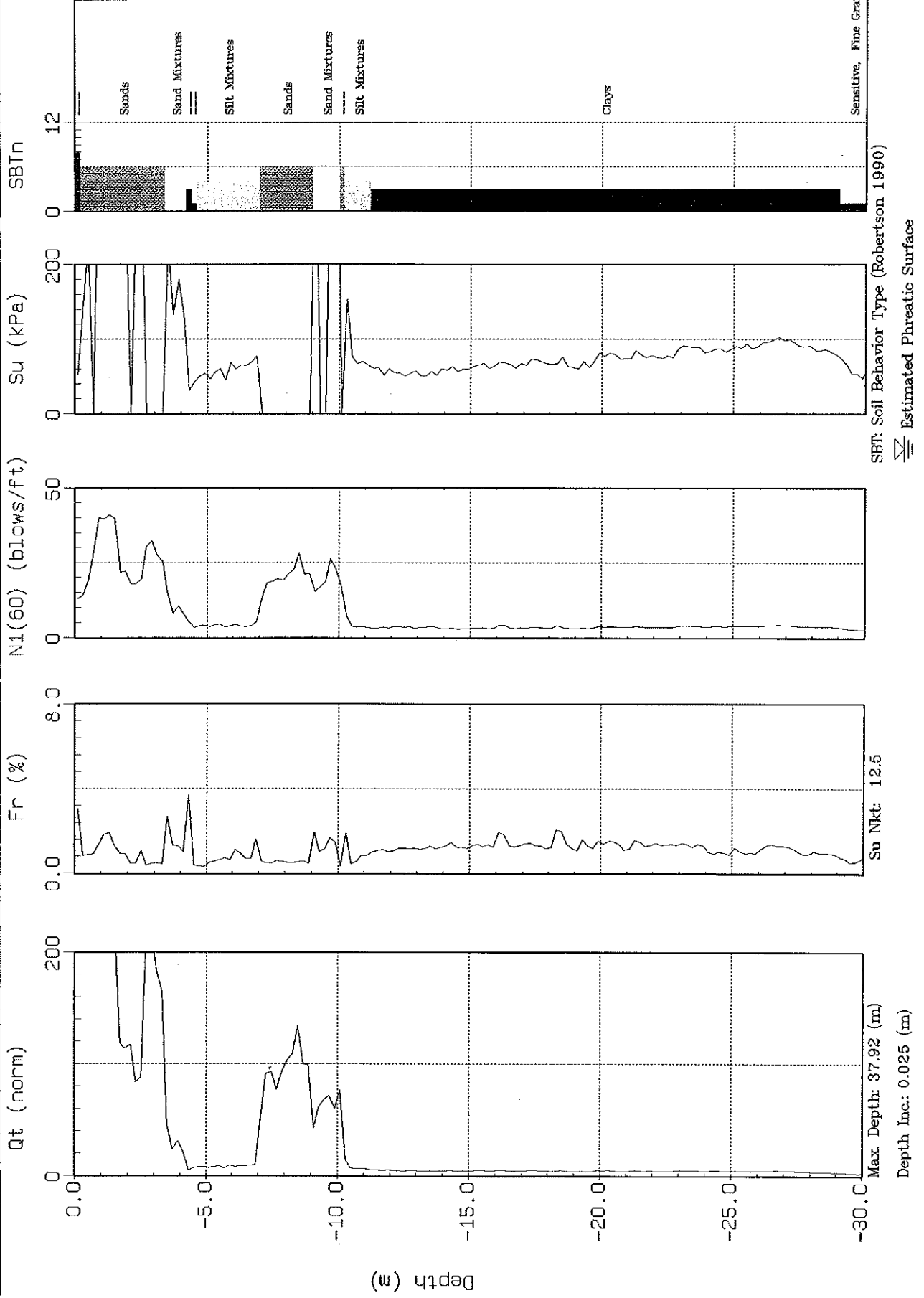
Thurber

Site: CPT04-04S

Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113

Date: 05:16:04 07:25





Thurber

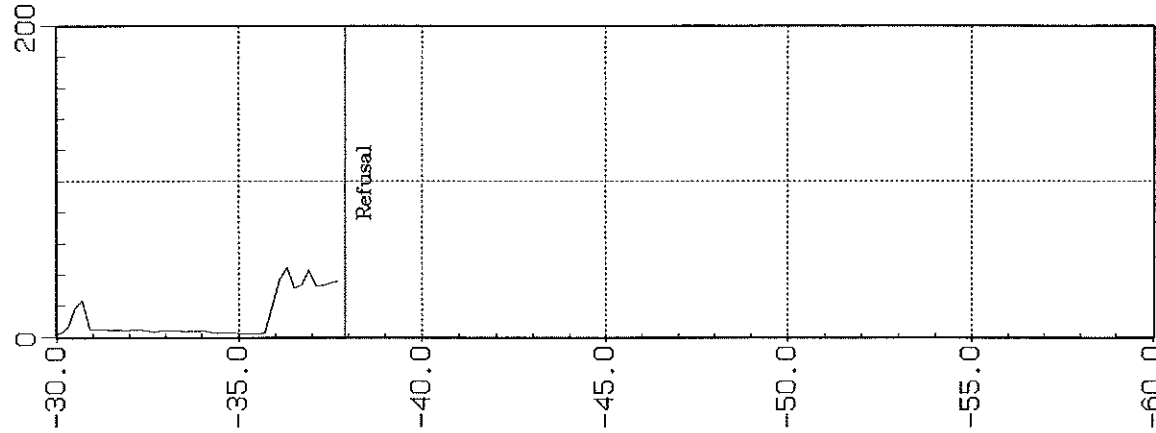
Site: CPT04-04S

Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113

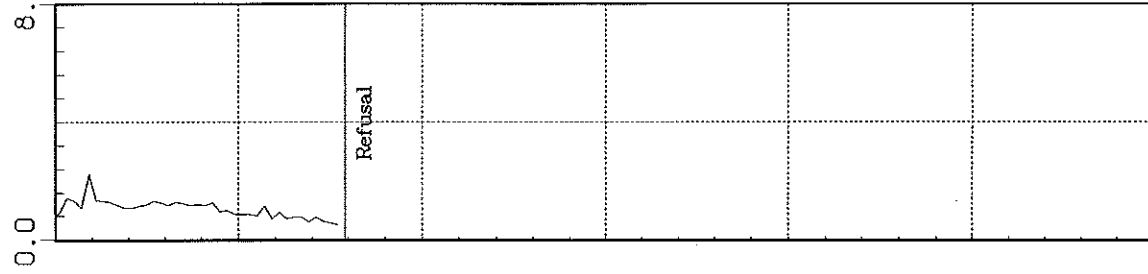
Date: 05:16:04 07:25

Qt (norm)



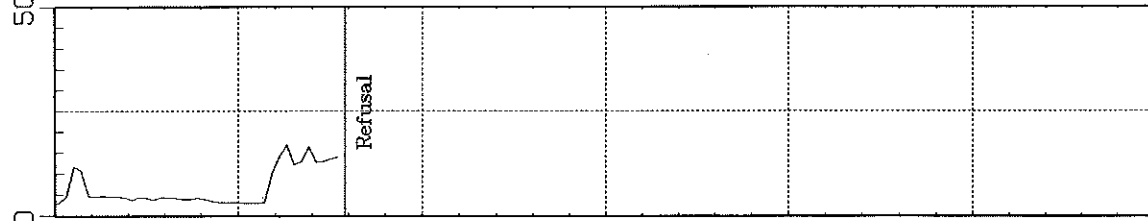
Max Depth: 37.92 (m)  
Depth Inc.: 0.025 (m)

Fr (%)

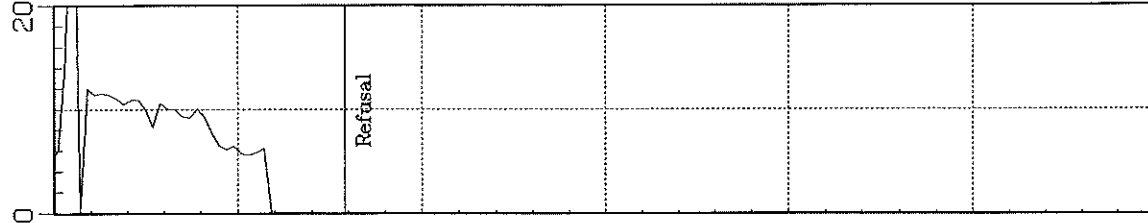


Su Nkt: 12.5

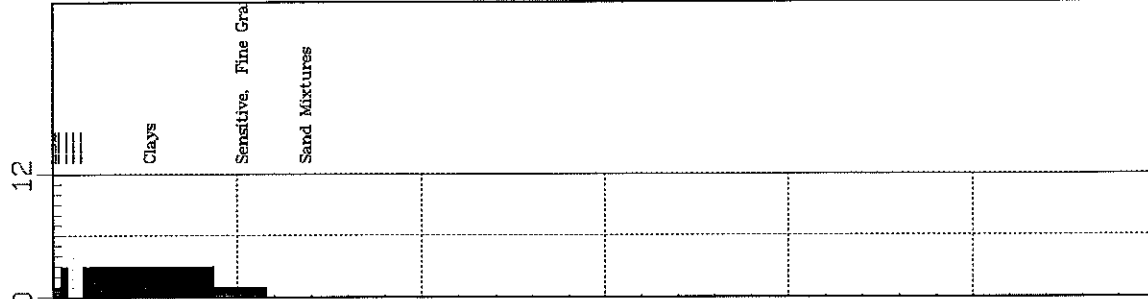
N1(60) (blows/ft)



Su (kPa)



SBTn



SBT: Soil Behavior Type (Robertson 1990)

Clays

Sensitive, Fine Gra

Sand Mixtures

# Appendix SCPTU

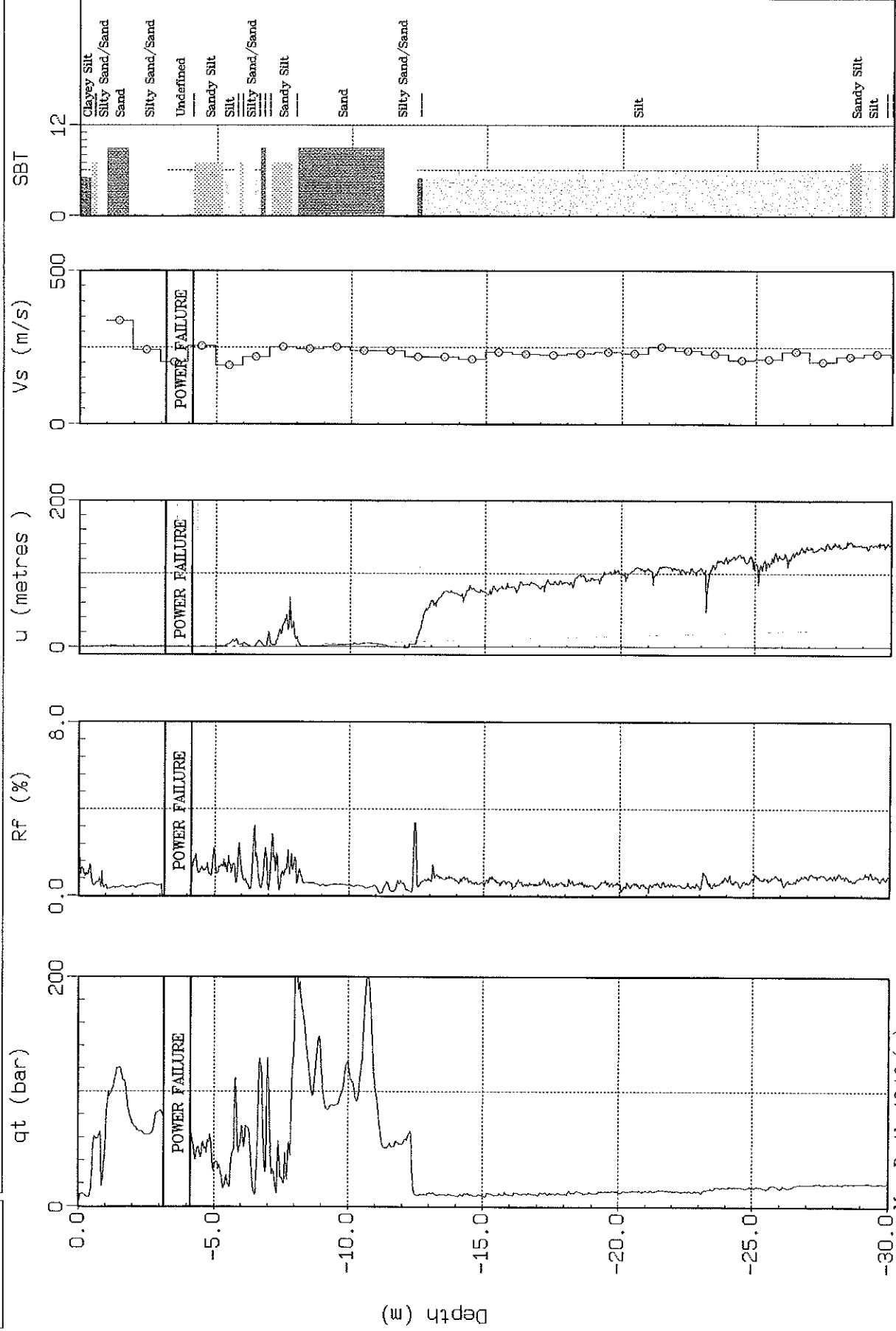
## Seismic Cone Penetration Tests



Thurber

Site: SCPT04-01N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:17:04 08:09



SBT: Soil Behavior Type (Robertson 1990)

Estimated Phreatic Surface

Max Depth: 46.10 (m)

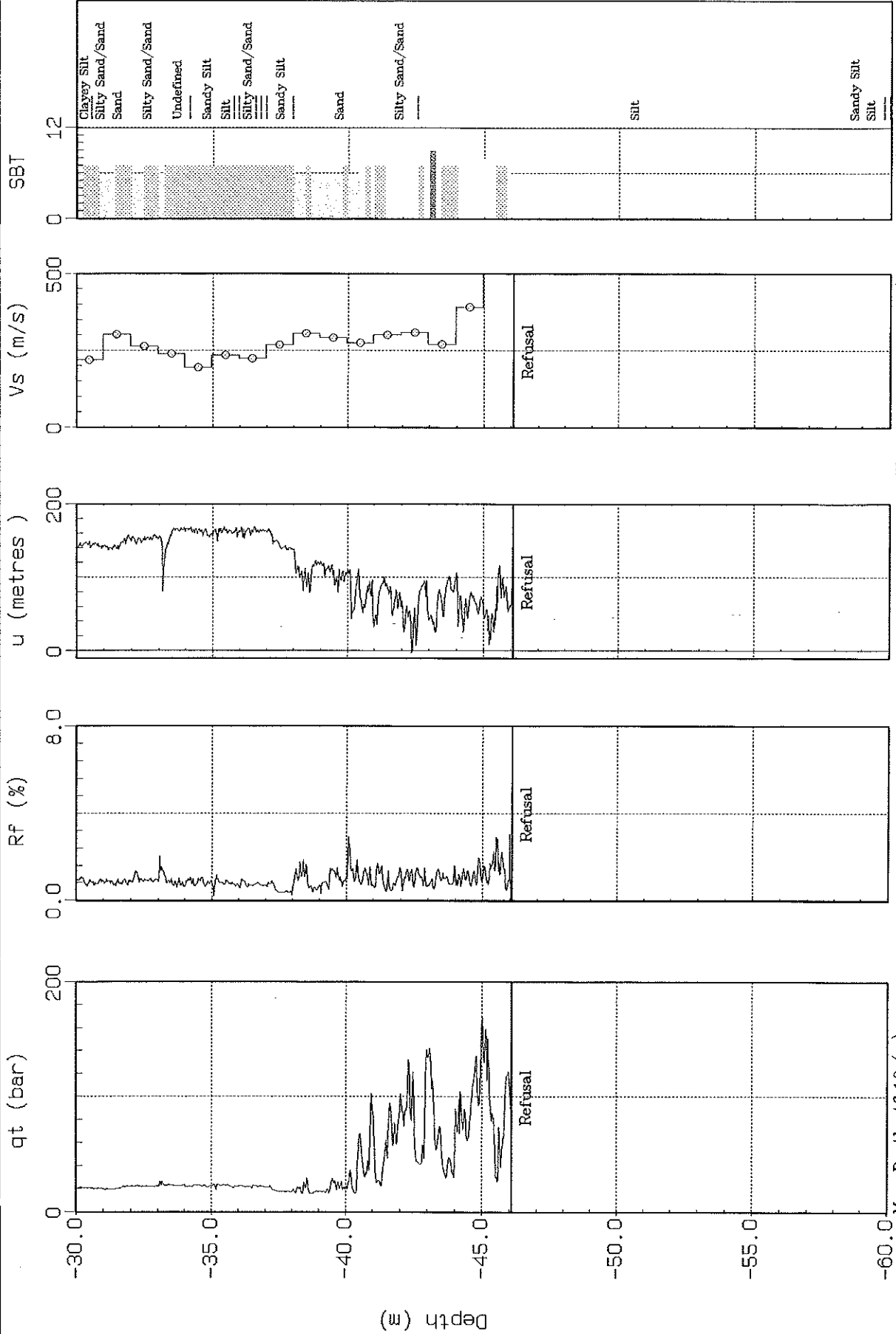
Depth Inc.: 0.025 (m)



Thurber

Site: SCPT04-01N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:17:04 08:09



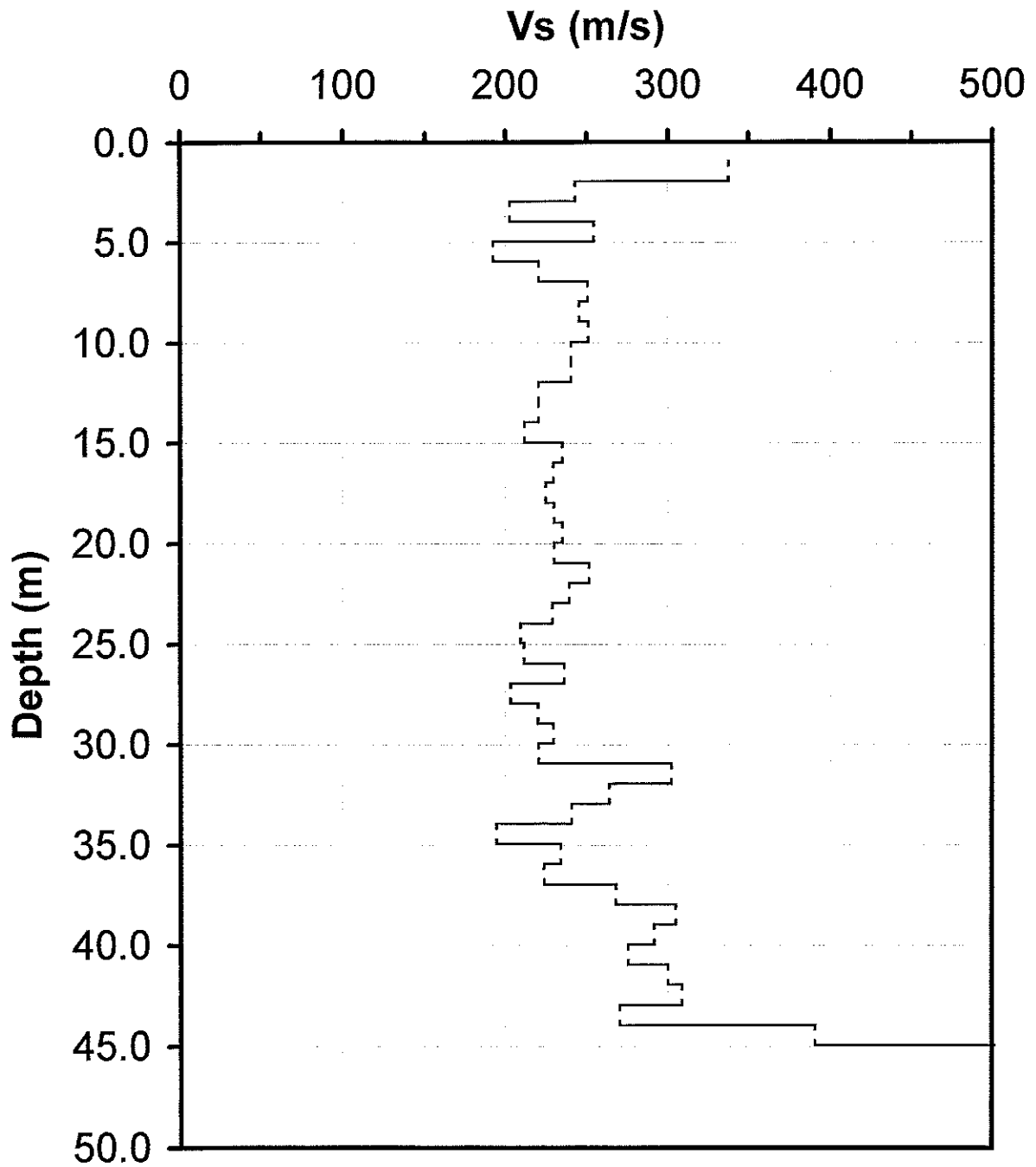
SBT: Soil Behavior Type (Robertson 1990)

Max Depth: 46.10 (m)  
Depth Inc.: 0.025 (m)





Job No: 04-166  
Client: Thurber Engineering Ltd.  
Location: Highway 69 4 Lane, Estaire  
Sounding: SCPT04-01N  
Sounding Date: May 17, 2004



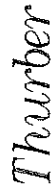


Job No.: 04-166  
Client: Thurber Engineering Ltd.  
Project: Highway 69 4 Lane, Estaire, Ontario  
Sounding: SCPT04-01N  
Date: 17-May-04

Seismic Source: BEAM  
Source Offset (m): 0.44  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

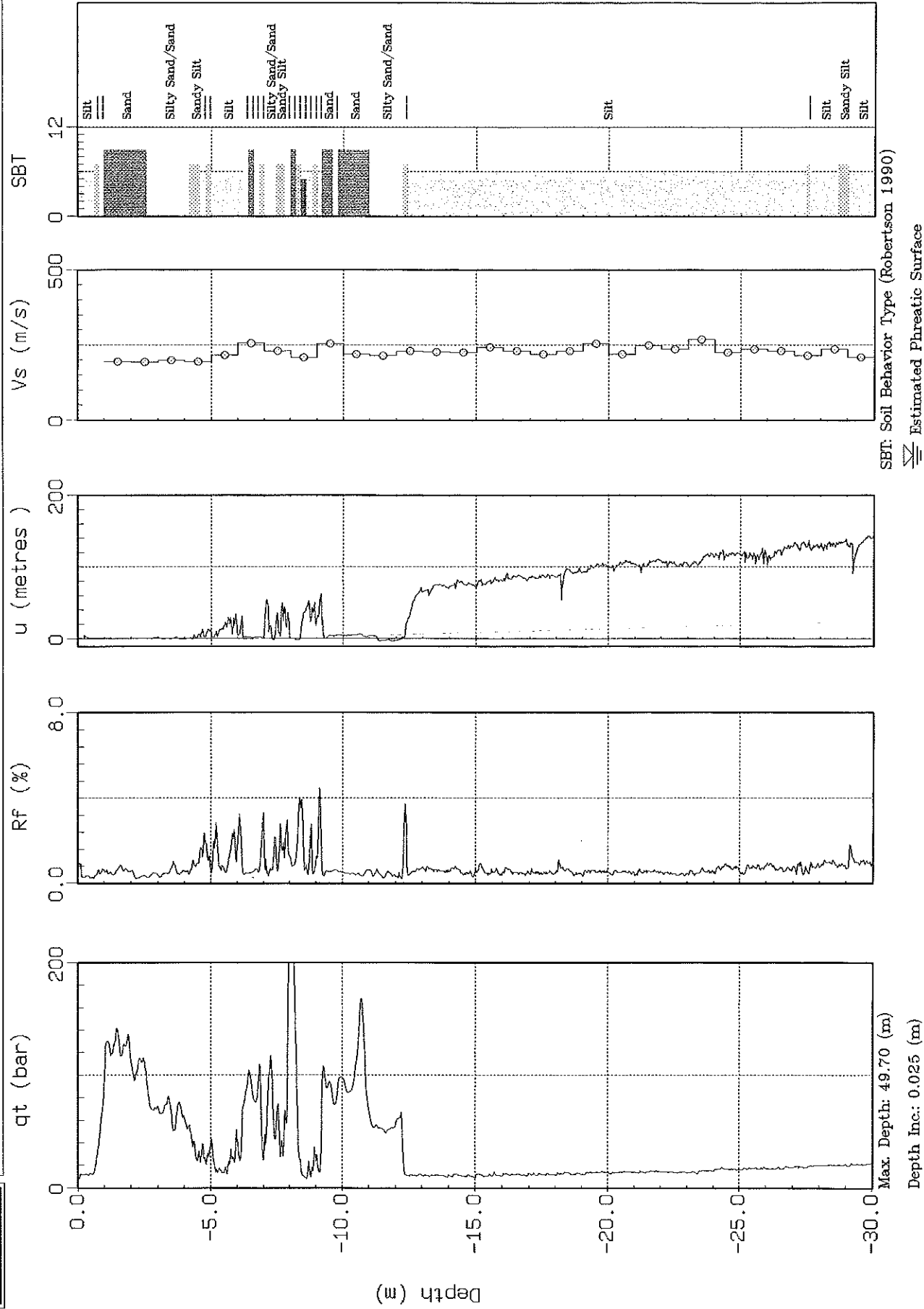
## SEISMIC

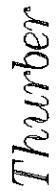
Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Depth Interval (m)	Time Interval (ms)	Vs (m/s)	Mid Layer Depth (m)
1.15	0.95	1.05				
2.15	1.95	2.00	0.95	2.82	338	1.45
3.15	2.95	2.98	0.98	4.05	243	2.45
4.15	3.95	3.97	0.99	4.90	202	3.45
5.15	4.95	4.97	1.00	3.91	254	4.45
6.15	5.95	5.97	1.00	5.18	192	5.45
7.15	6.95	6.96	1.00	4.53	220	6.45
8.15	7.95	7.96	1.00	3.98	251	7.45
9.15	8.95	8.96	1.00	4.07	245	8.45
10.15	9.95	9.96	1.00	3.98	251	9.45
11.15	10.95	10.96	1.00	4.16	240	10.45
12.15	11.95	11.96	1.00	4.16	240	11.45
13.15	12.95	12.96	1.00	4.54	220	12.45
14.15	13.95	13.96	1.00	4.54	220	13.45
15.15	14.95	14.96	1.00	4.73	211	14.45
16.15	15.95	15.96	1.00	4.26	235	15.45
17.15	16.95	16.96	1.00	4.36	229	16.45
18.15	17.95	17.96	1.00	4.45	225	17.45
19.15	18.95	18.96	1.00	4.35	230	18.45
20.15	19.95	19.95	1.00	4.26	235	19.45
21.15	20.95	20.95	1.00	4.35	230	20.45
22.15	21.95	21.95	1.00	3.98	251	21.45
23.15	22.95	22.95	1.00	4.18	239	22.45
24.15	23.95	23.95	1.00	4.37	229	23.45
25.15	24.95	24.95	1.00	4.78	209	24.45
26.15	25.95	25.95	1.00	4.74	211	25.45
27.15	26.95	26.95	1.00	4.24	236	26.45
28.15	27.95	27.95	1.00	4.93	203	27.45
29.15	28.95	28.95	1.00	4.55	220	28.45
30.15	29.95	29.95	1.00	4.36	229	29.45
31.15	30.95	30.95	1.00	4.54	220	30.45
32.15	31.95	31.95	1.00	3.31	302	31.45
33.15	32.95	32.95	1.00	3.79	264	32.45
34.15	33.95	33.95	1.00	4.16	240	33.45
35.15	34.95	34.95	1.00	5.14	195	34.45
36.15	35.95	35.95	1.00	4.28	234	35.45
37.15	36.95	36.95	1.00	4.47	224	36.45
38.15	37.95	37.95	1.00	3.73	268	37.45
39.15	38.95	38.95	1.00	3.28	305	38.45
40.15	39.95	39.95	1.00	3.43	292	39.45
41.15	40.95	40.95	1.00	3.63	275	40.45
42.15	41.95	41.95	1.00	3.33	300	41.45
43.15	42.95	42.95	1.00	3.24	309	42.45
44.15	43.95	43.95	1.00	3.70	270	43.45
45.15	44.95	44.95	1.00	2.56	391	44.45
46.10	45.90	45.90	0.95	1.72	552	45.43



Location: HWY 69 4L ESTAIRS

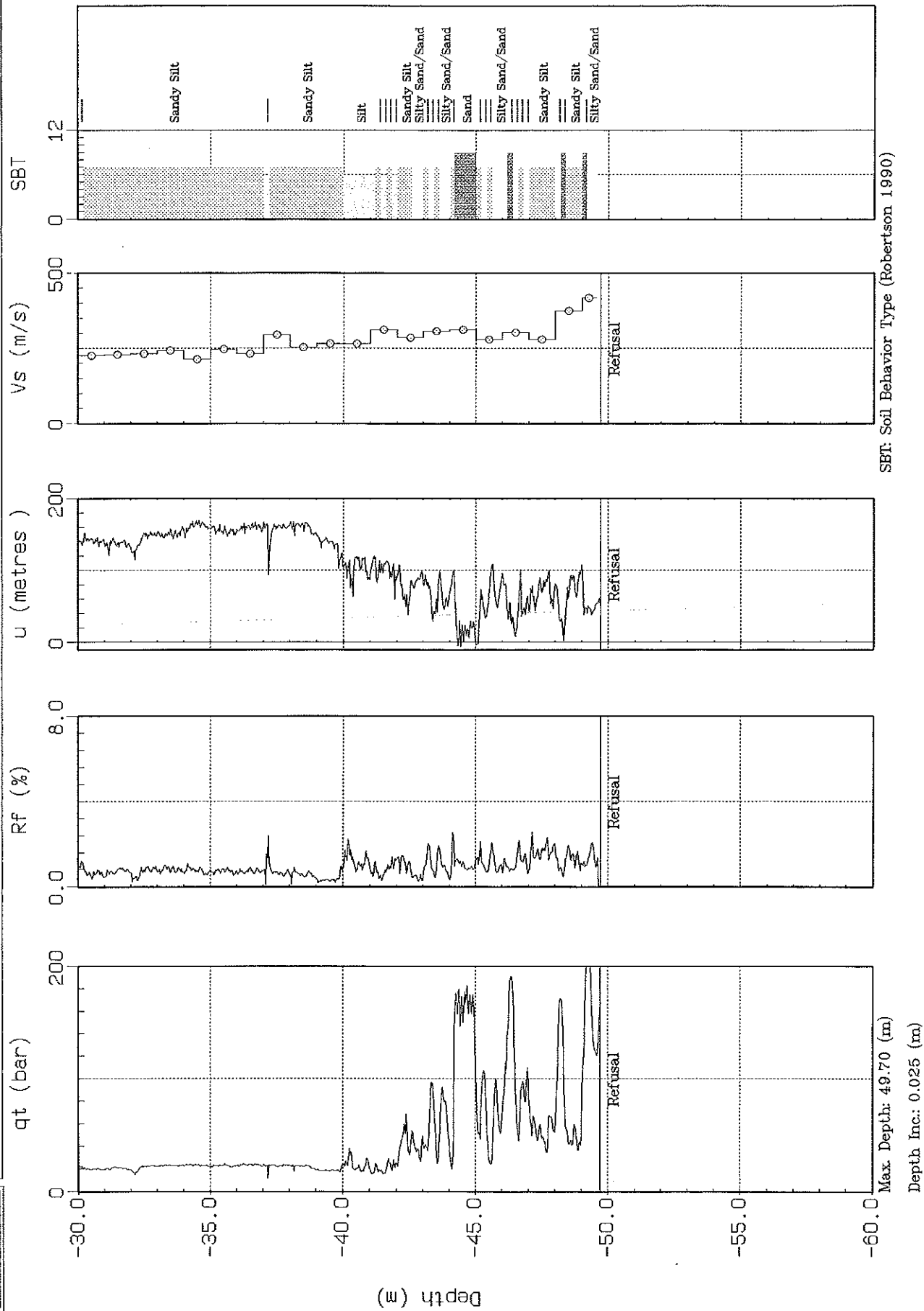
Date: 05:17:04 12:50





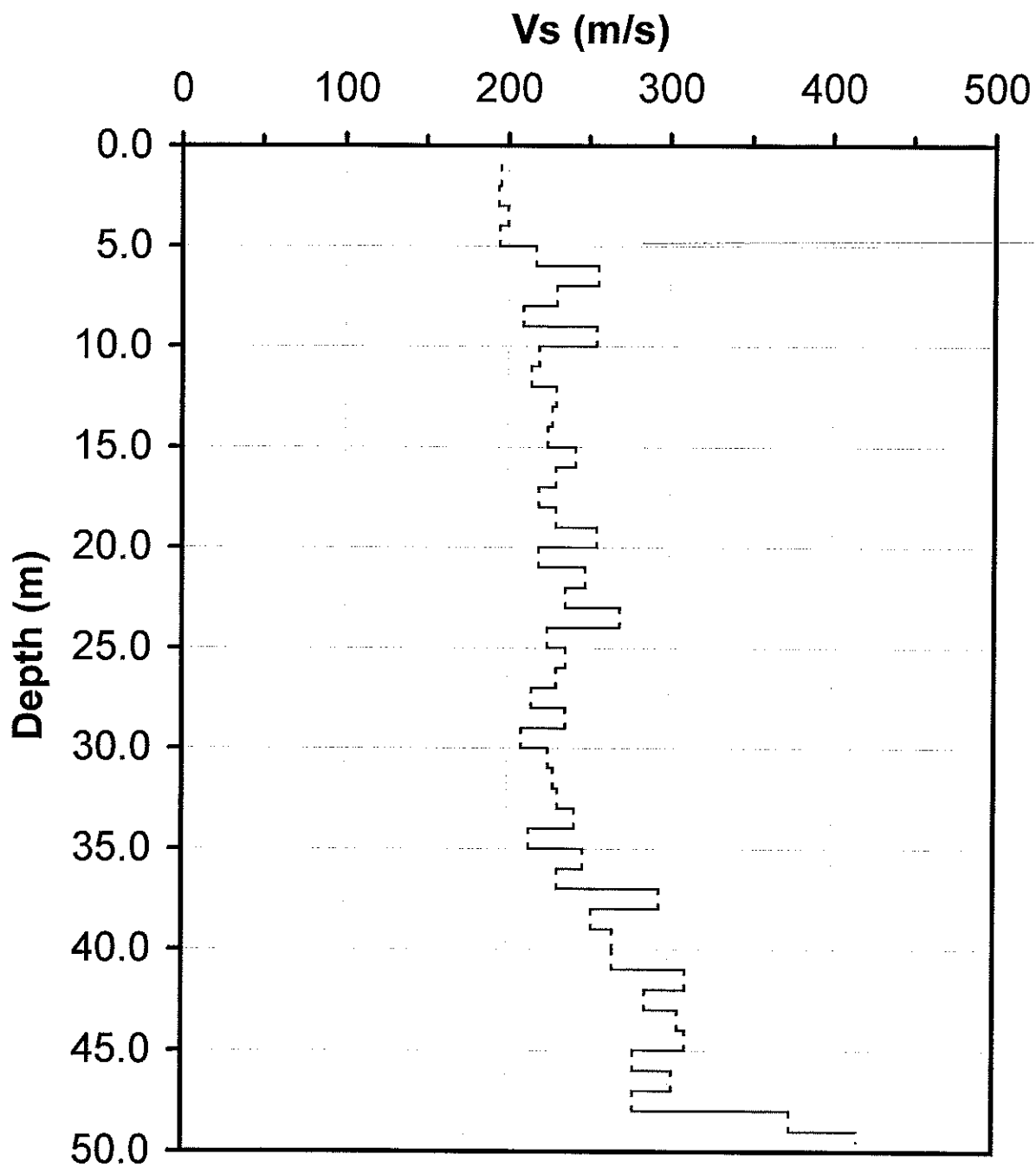
Cone: 20 Ton St 113

06:05:17:04 12:50





Job No: 04-166  
Client: Thurber Engineering Ltd.  
Location: Highway 69 4 Lane, Estaire  
Sounding: SCPT04-01S  
Sounding Date: May 17, 2004





Job No.: 04-166  
Client: Thurber Engineering Ltd.  
Project: Highway 69 4 Lane, Estaire, Ontario  
Sounding: SCPT04-01S  
Date: 17-May-04

Seismic Source: BEAM  
Source Offset (m): 0.48  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

## SEISMIC

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Depth Interval (m)	Time Interval (ms)	Vs (m/s)	Mid Layer Depth (m)
1.20	1.00	1.11				
2.20	2.00	2.06	0.95	4.85	195	1.50
3.20	3.00	3.04	0.98	5.06	194	2.50
4.20	4.00	4.03	0.99	4.96	200	3.50
5.20	5.00	5.02	0.99	5.11	195	4.50
6.20	6.00	6.02	1.00	4.59	217	5.50
7.20	7.00	7.02	1.00	3.90	256	6.50
8.20	8.00	8.01	1.00	4.34	230	7.50
9.20	9.00	9.01	1.00	4.77	209	8.50
10.20	10.00	10.01	1.00	3.92	255	9.50
11.20	11.00	11.01	1.00	4.56	219	10.50
12.20	12.00	12.01	1.00	4.66	214	11.50
13.20	13.00	13.01	1.00	4.35	230	12.50
14.20	14.00	14.01	1.00	4.40	227	13.50
15.20	15.00	15.01	1.00	4.45	225	14.50
16.20	16.00	16.01	1.00	4.13	242	15.50
17.20	17.00	17.01	1.00	4.35	230	16.50
18.20	18.00	18.01	1.00	4.56	219	17.50
19.20	19.00	19.01	1.00	4.35	230	18.50
20.20	20.00	20.01	1.00	3.92	255	19.50
21.20	21.00	21.01	1.00	4.56	219	20.50
22.20	22.00	22.01	1.00	4.03	248	21.50
23.20	23.00	23.01	1.00	4.24	236	22.50
24.20	24.00	24.00	1.00	3.71	269	23.50
25.20	25.00	25.00	1.00	4.45	225	24.50
26.20	26.00	26.00	1.00	4.24	236	25.50
27.20	27.00	27.00	1.00	4.35	230	26.50
28.20	28.00	28.00	1.00	4.66	215	27.50
29.20	29.00	29.00	1.00	4.24	236	28.50
30.20	30.00	30.00	1.00	4.79	209	29.50
31.20	31.00	31.00	1.00	4.44	225	30.50
32.20	32.00	32.00	1.00	4.38	228	31.50
33.20	33.00	33.00	1.00	4.33	231	32.50
34.20	34.00	34.00	1.00	4.14	242	33.50
35.20	35.00	35.00	1.00	4.69	213	34.50
36.20	36.00	36.00	1.00	4.05	247	35.50
37.20	37.00	37.00	1.00	4.33	231	36.50
38.20	38.00	38.00	1.00	3.40	294	37.50
39.20	39.00	39.00	1.00	3.96	253	38.50
40.20	40.00	40.00	1.00	3.77	265	39.50
41.20	41.00	41.00	1.00	3.77	265	40.50
42.20	42.00	42.00	1.00	3.22	311	41.50
43.20	43.00	43.00	1.00	3.50	286	42.50
44.20	44.00	44.00	1.00	3.27	306	43.50
45.20	45.00	45.00	1.00	3.22	311	44.50
46.20	46.00	46.00	1.00	3.59	279	45.50
47.20	47.00	47.00	1.00	3.31	302	46.50
48.20	48.00	48.00	1.00	3.59	279	47.50
49.20	49.00	49.00	1.00	2.67	375	48.50
49.70	49.50	49.50	0.50	1.20	417	49.25

# Appendix PPD

## Pore Pressure Dissipation Tests



Job No: 04-166  
 Client: Thurber Engineering Ltd.  
 Project: Highway 69-4 Lane, Estaire, Ontario  
 Date: May 15 - 18, 2004

### PPD SUMMARY

CPT Sounding	Filename	Test Depth (m)	Equilibrium Pore Pressure (Ueq, m)*	Assumed Water Table (m)**	T <sub>50</sub> (s)***	C <sub>h</sub> (cm <sup>2</sup> /min)****
SCPT04-01N	166C01N	5.15	0.8	4.4	NA	NA
		10.15	3.5	6.6	NA	NA
		23.15	NA	6.6	308	2.32
		33.15	NA	6.6	989	0.72
SCPT04-01S	166C01S	10.20	4.5	5.7	NA	NA
		18.20	12.5	5.7	478	1.50
		29.20	23.5	5.7	3044	0.24
		37.20	31.5	5.7	2285	0.31
		46.70	41.5	5.7	48	15.00
		49.70	44.5	5.2	NA	NA
CPT04-02N	166C02N	7.15	1.6	5.6	NA	NA
		20.15	NA	5.6	627	1.14
		34.15	NA	5.6	2239	0.32
		43.13	NA	5.6	53	13.57
CPT04-02S	166C02S	7.18	2.8	4.4	NA	NA
		28.20	NA	4.4	2429	0.30
		49.68	44.2	5.5	NA	NA
CPT04-03N	166C03N	8.17	1.7	6.5	29	24.45
		19.97	NA	6.5	297	2.41
		33.97	NA	6.5	1985	0.36
		42.90	NA	6.5	165	4.35
		46.35	39.3	7.0	NA	NA
CPT04-03S	166C03S	10.30	3.0	7.3	NA	NA
		27.92	NA	7.3	2249	0.32
		40.25	NA	7.3	448	1.60
		41.25	NA	7.3	200	3.58
CPT04-04N	166C04N	8.28	2.4	5.9	NA	NA
		18.27	NA	5.9	236	3.04
CPT04-04S	166C04S	7.05	3.1	4.0	NA	NA
		24.05	NA	4.0	1873	0.38

\* Equilibrium pore pressure estimated from complete dissipation test (T100).

\*\* Assumed Water Table is used to estimate Ueq for T50 calculation. It is calculated from complete dissipation tests in same hole.

\*\*\* T<sub>50</sub> is calculated based on U initial and estimated equilibrium pore pressure

\*\*\*\* C<sub>h</sub> calculated based on Robertson et al., 1992



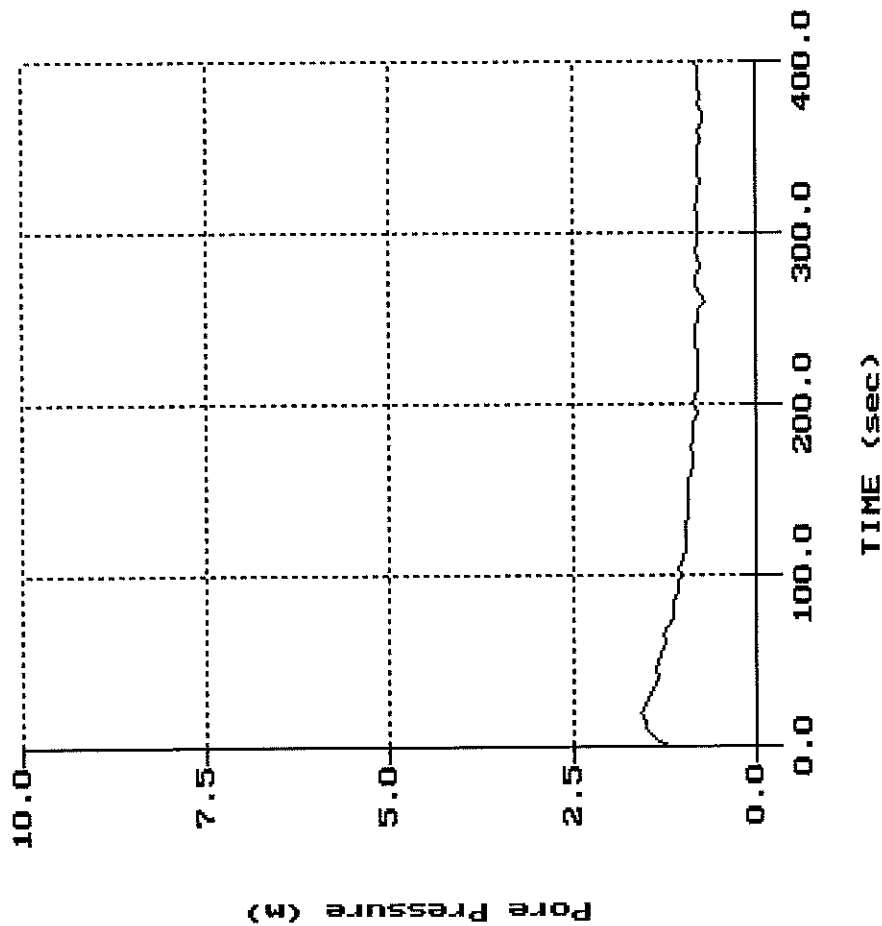
# Thurber

Hole: SCPT04-01N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:17:04 08:09

File: 166C01N.PPD  
Depth (m): 5.15  
Depth (ft): 16.90  
Duration: 400.0s  
U-min: 0.70 260.0s  
U-max: 1.59 20.0s

## PORE PRESSURE DISSIPATION RECORD

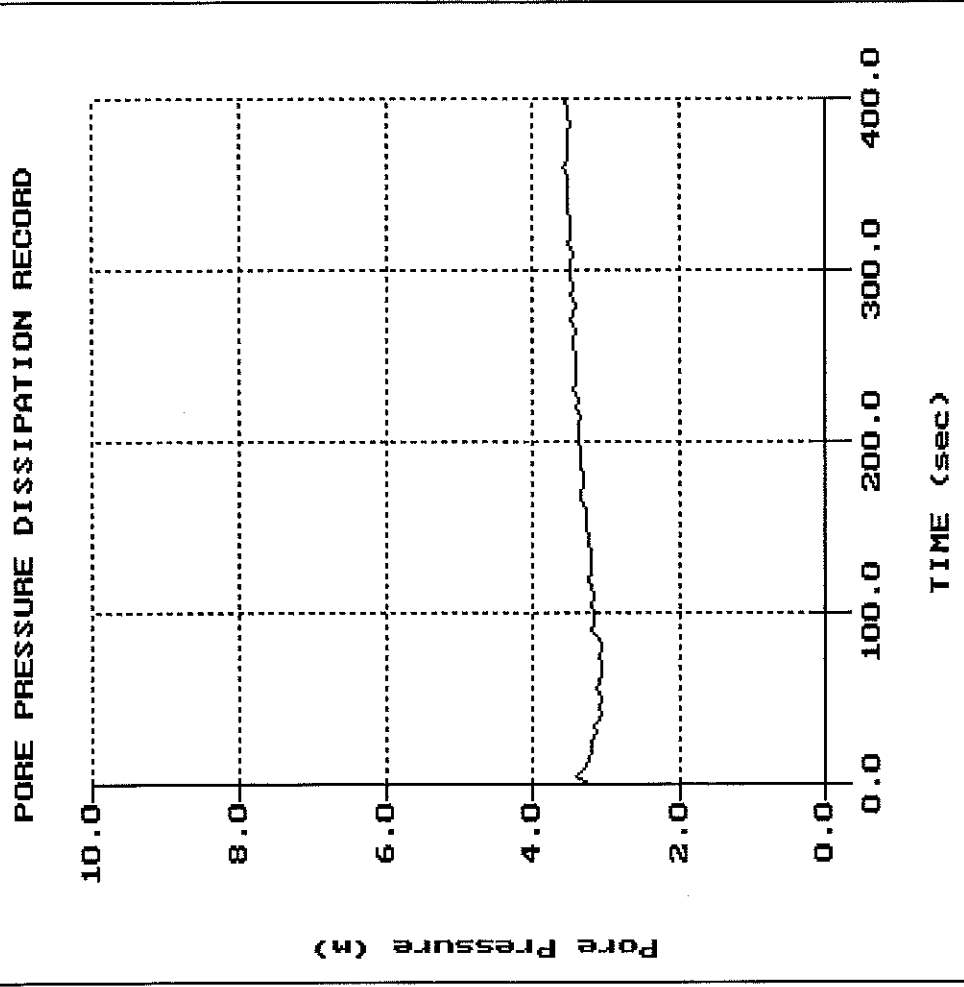


# Thurber

Hole: SCPT04-01N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:17:04 08:09

File: 166C01N.PPD  
Depth (m): 10.15  
Duration: 400.0s  
U-min: 3.05 80.0s  
U-max: 3.56 400.0s

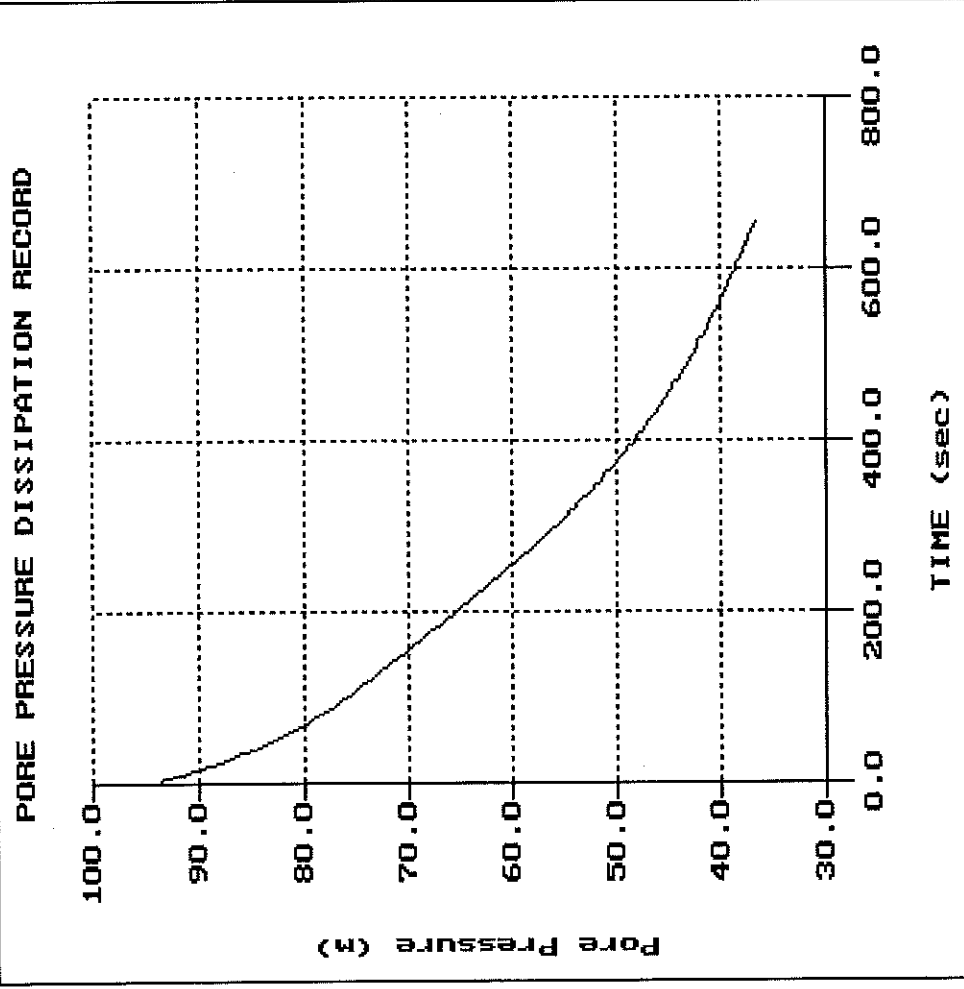


# Thurber

Hole: SCPT04-01N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:17:04 08:09

File: 166C01N.PPD  
Depth (m): 23.15  
Duration: 75.95  
U-min: 36.67 655.0s  
U-max: 93.59 0.0s

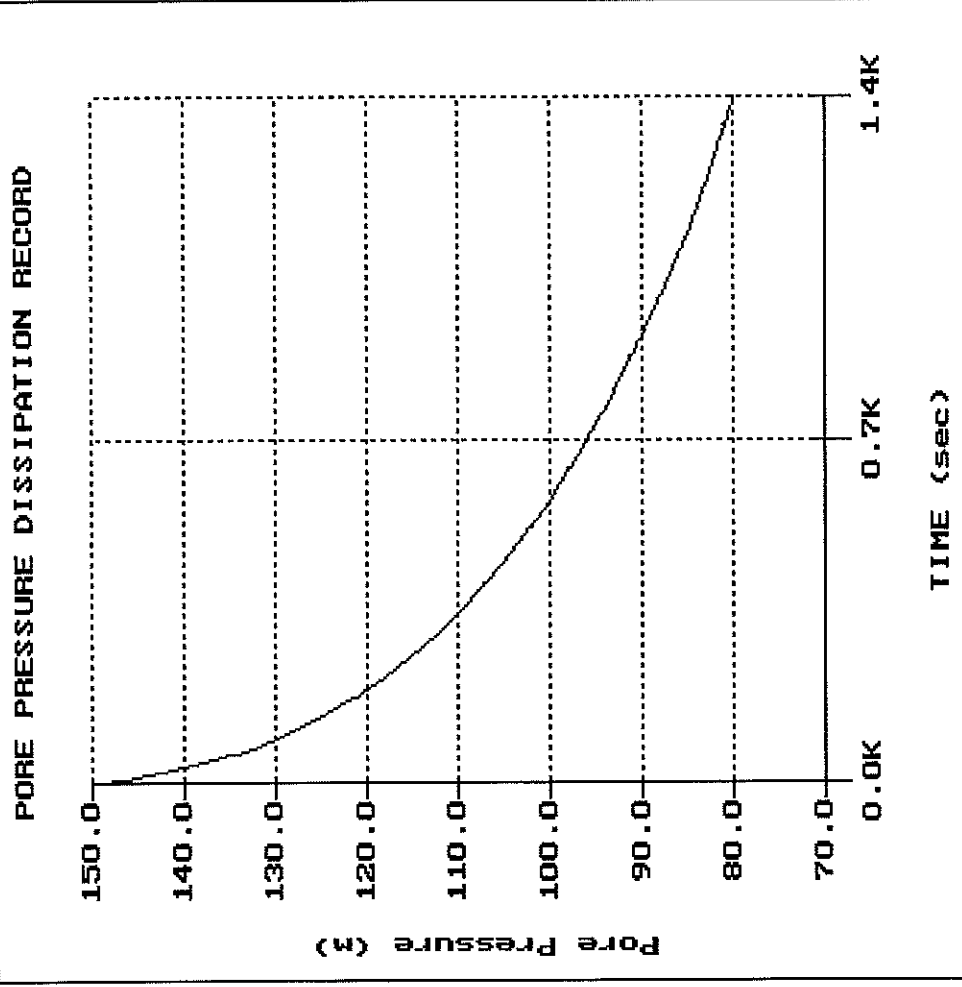


# Thurber

Hole: SCPT04-01N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:17:04 08:09

File: 166C001N.PPD  
Depth (m): 33.15  
Duration: 108.76  
U-min: 79.93 1395.0s  
U-max: 149.69 0.0s



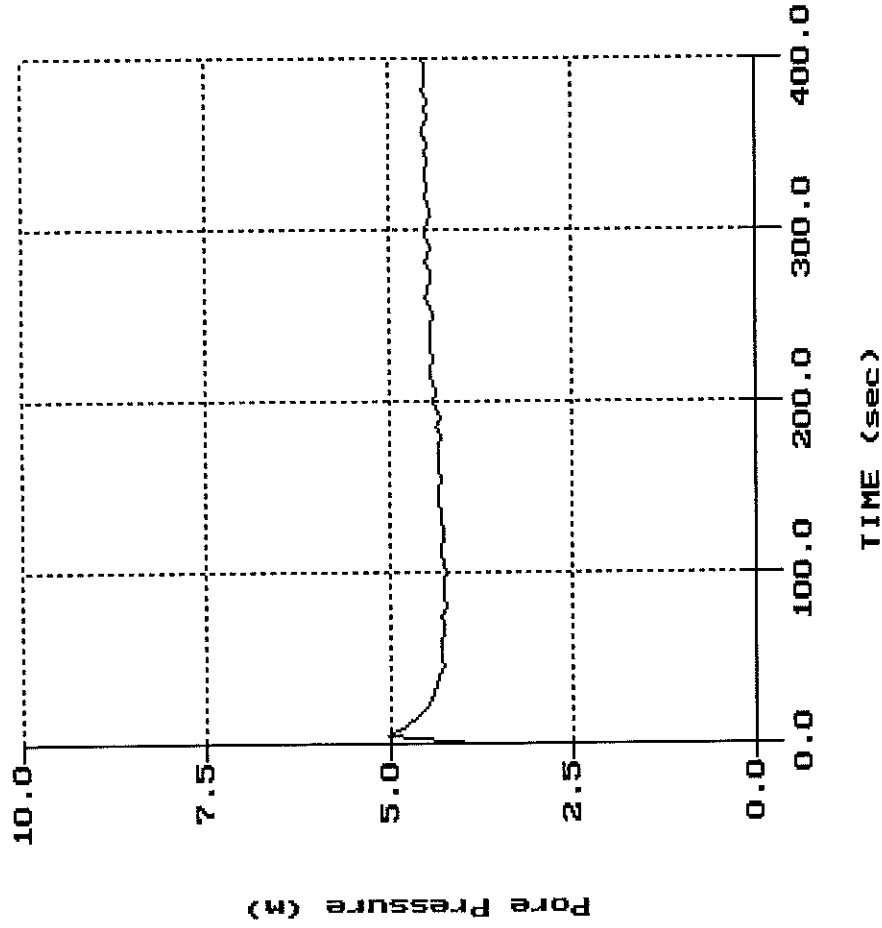
# Thurber

Hole: SCPT04-01S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:17:04 12:50

File: 166C01S.PPD  
Depth (m): 10.20  
(ft): 33.46  
Duration: 400.0s  
U-min: 3.80 0.0s  
U-max: 5.03 5.0s

## PORE PRESSURE DISSIPATION RECORD



# Thurber

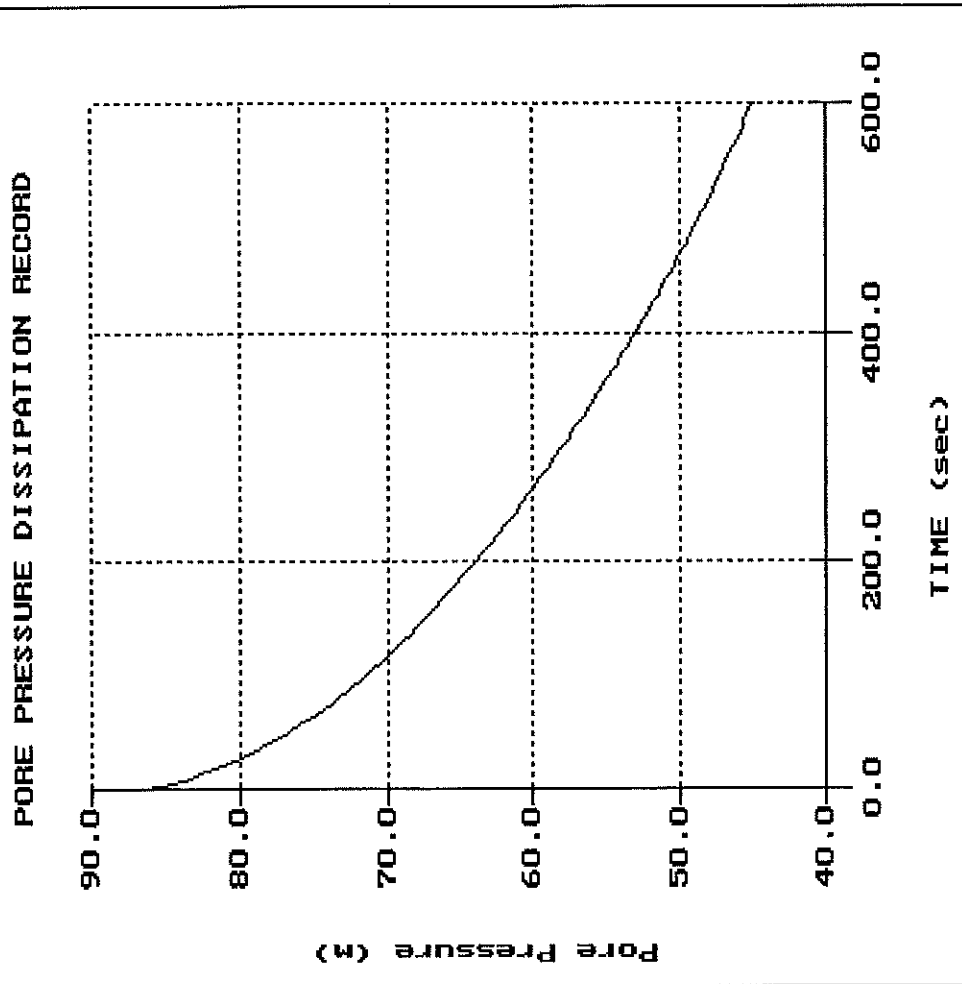
Hole: SCPT04-01S

Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113

Date: 05:17:04 12:50

File: 166C01S.PPD  
Depth (m): 18.20  
Duration (ft): 59.71  
U-min: 44.94 600.0s  
U-max: 86.38 0.0s

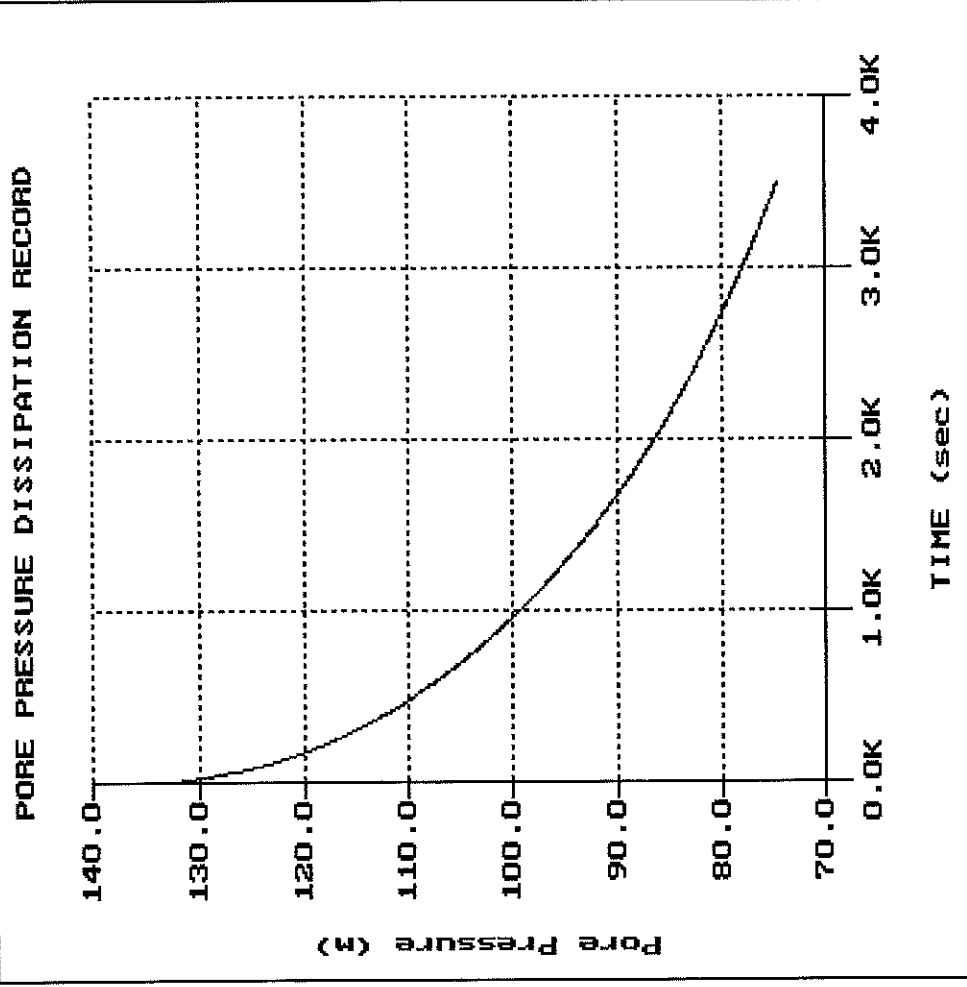


Thurber

Hole: SCPT04-01S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:17:04 12:50

File: 166C01S.PPD  
Depth (m): 29.20  
Duration (ft): 95.80  
U-min: 74.59 3495.0s  
U-max: 131.78 5.0s

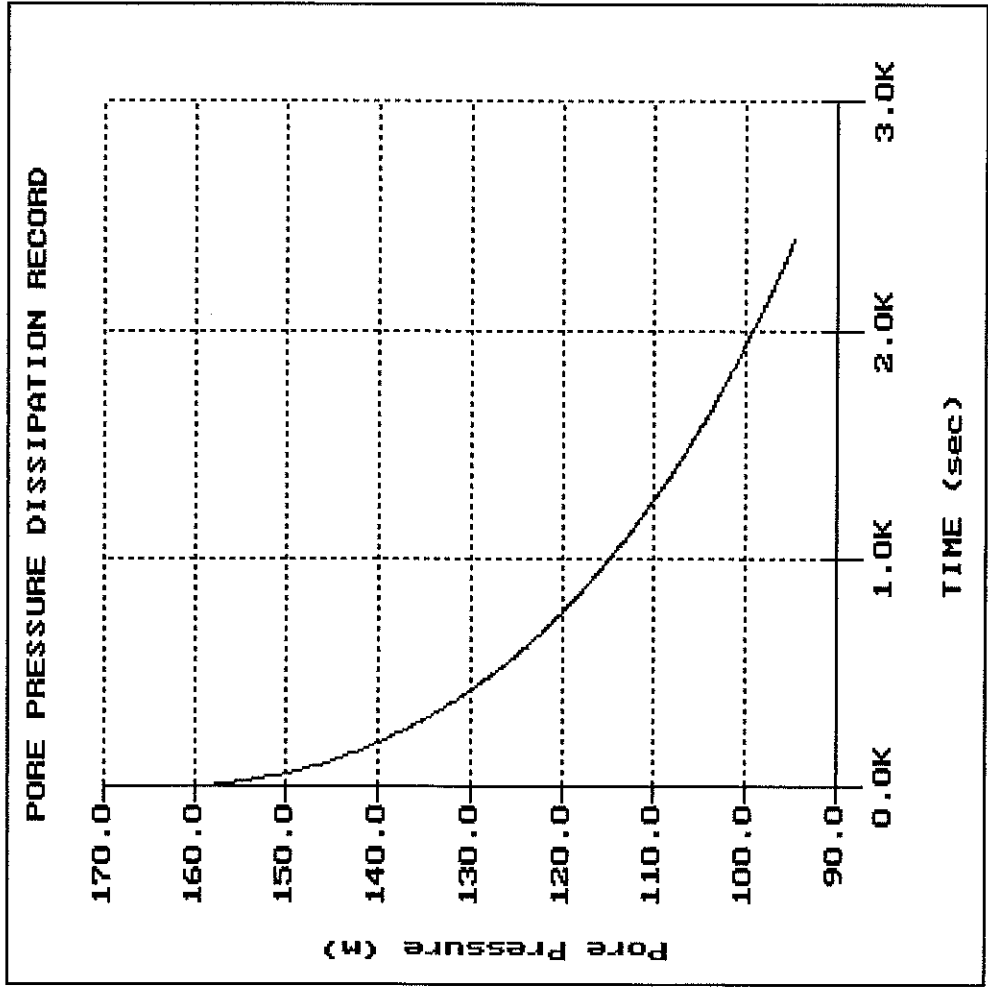


# Thurber

Hole: SCPT04-01S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:17:04 12:50

File: 166C01S.PPD  
Depth (m): 37.20  
(ft): 122.05  
Duration: 2400.0s  
U-min: 94.61 2400.0s  
U-max: 160.14 0.0s



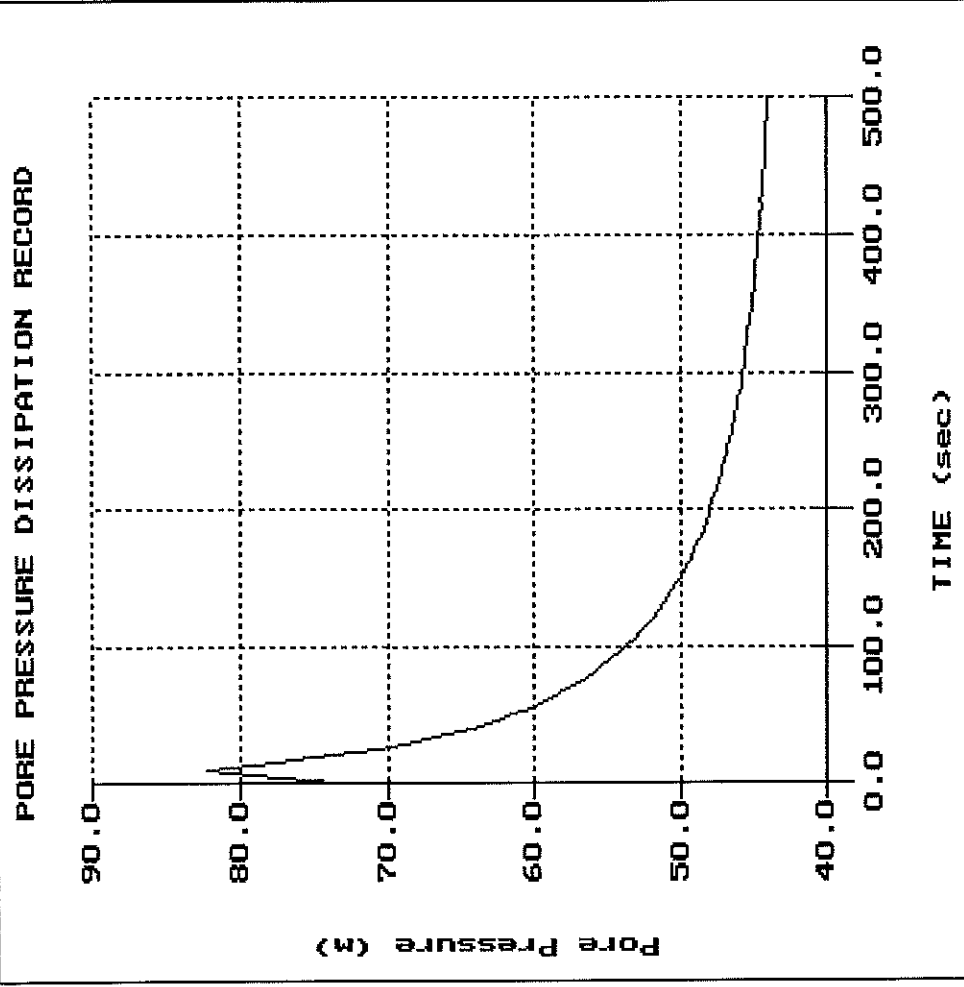


# Thurber

Hole: SCPT04-01S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:17:04 12:50

File: 166C01S.PPD  
Depth (m): 46.70  
(ft): 153.22  
Duration: 500.0s  
U-Min: 43.97 500.0s  
U-Max: 82.26 10.0s



Thurber

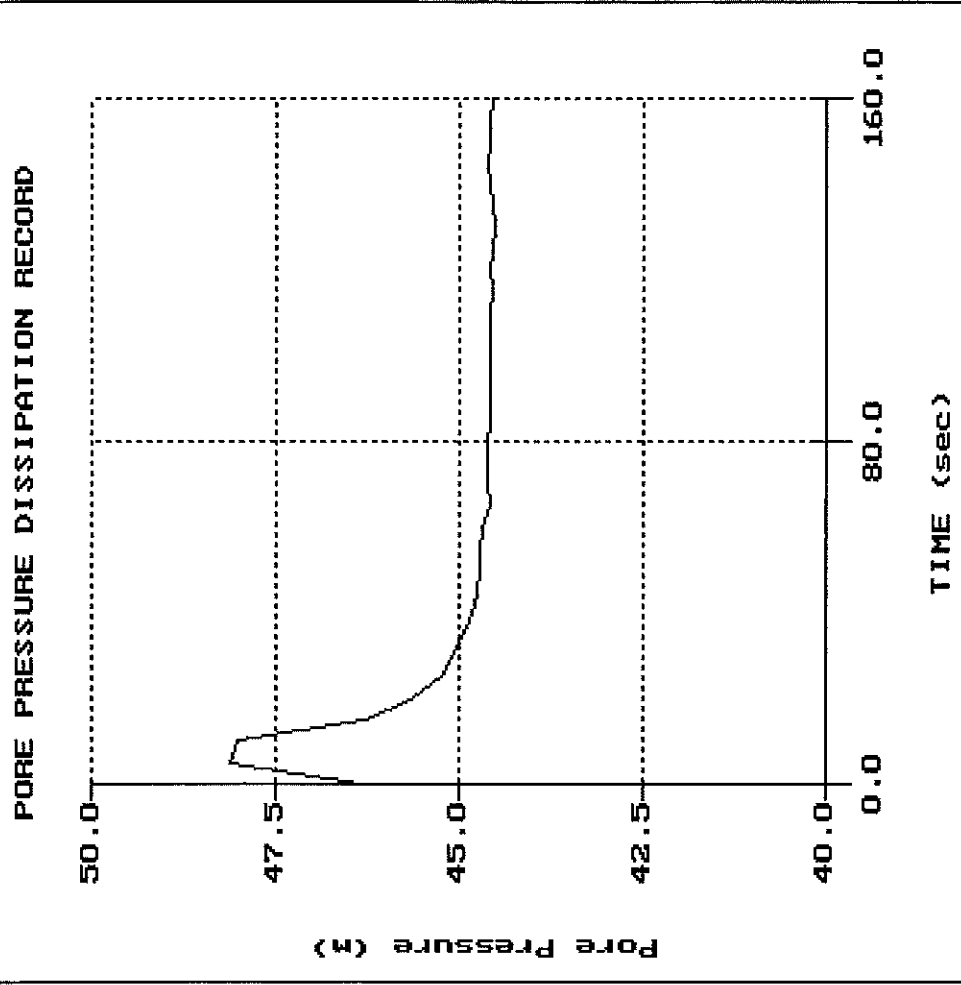
Hole: SCPT04-01S

Cone: 20 Ton St 113

Location: HWY69 4L ESTAIRE

Date: 05:17:04 12:50

File: 166C01S.PPD  
Depth (m): 49.70  
Duration (ft): 163.06  
U-min: 44.51 130.0s  
U-max: 48.13 5.0s



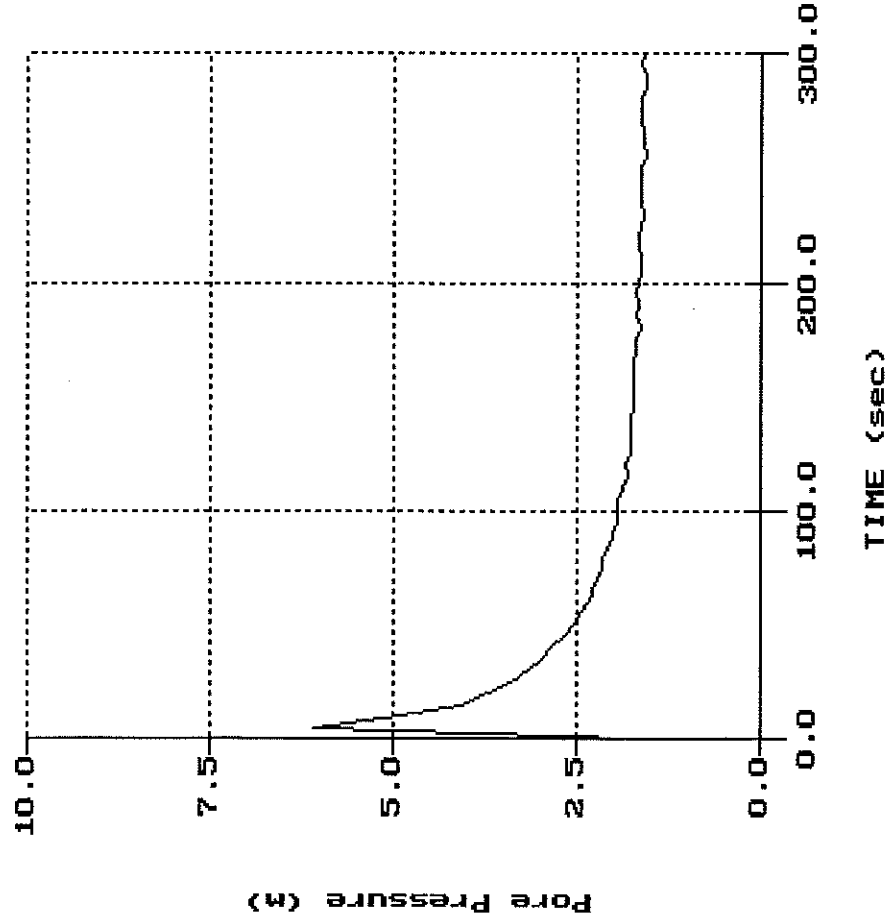
# Thurber

Hole: CPT04-02N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:18:04 11:22

File: 166C02N.PPD  
Depth (m): 7.15  
Depth (ft): 23.46  
Duration: 300.0s  
U-min: 1.57 290.0s  
U-max: 6.08 5.0s

## PORE PRESSURE DISSIPATION RECORD

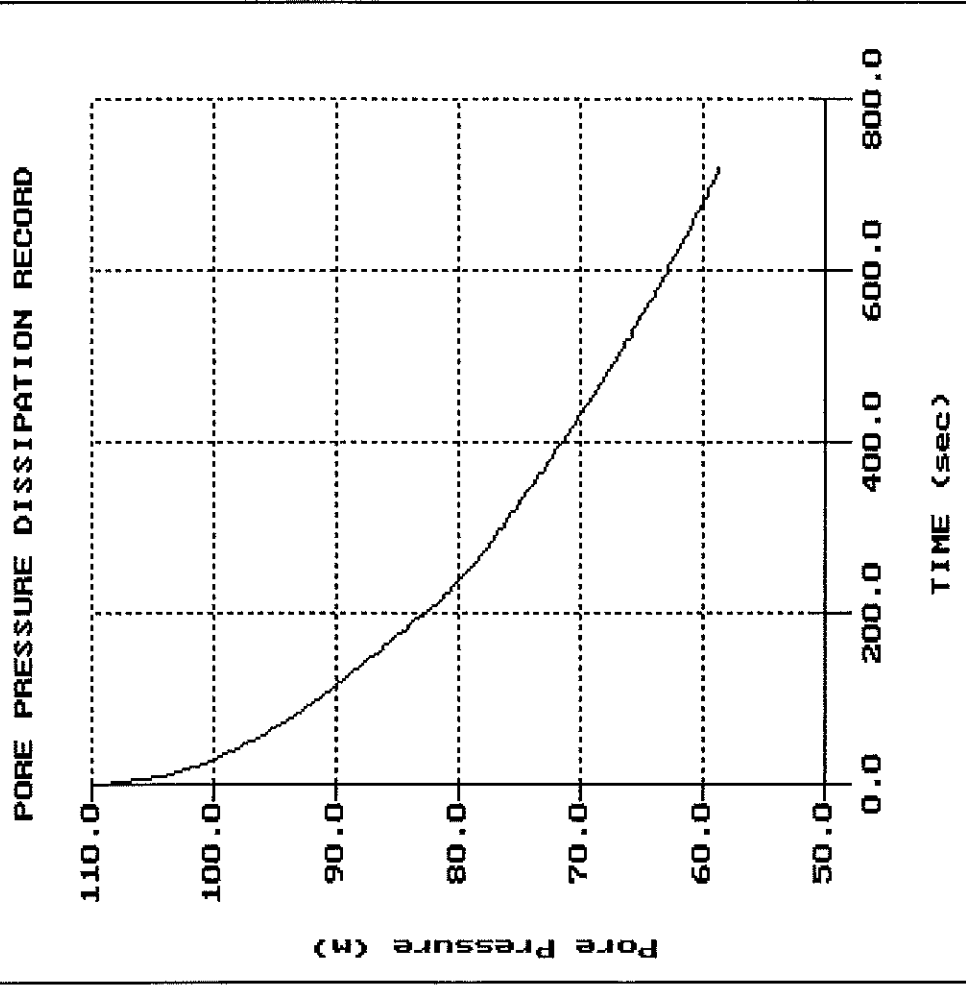


# Thurber

Hole: CPT04-02N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:18:04 11:22

File: 166C02N.PPD  
Depth (m): 20.15  
Depth (ft): 66.11  
Duration: 720.0s  
U-min: 58.67 720.0s  
U-max: 109.38 0.0s



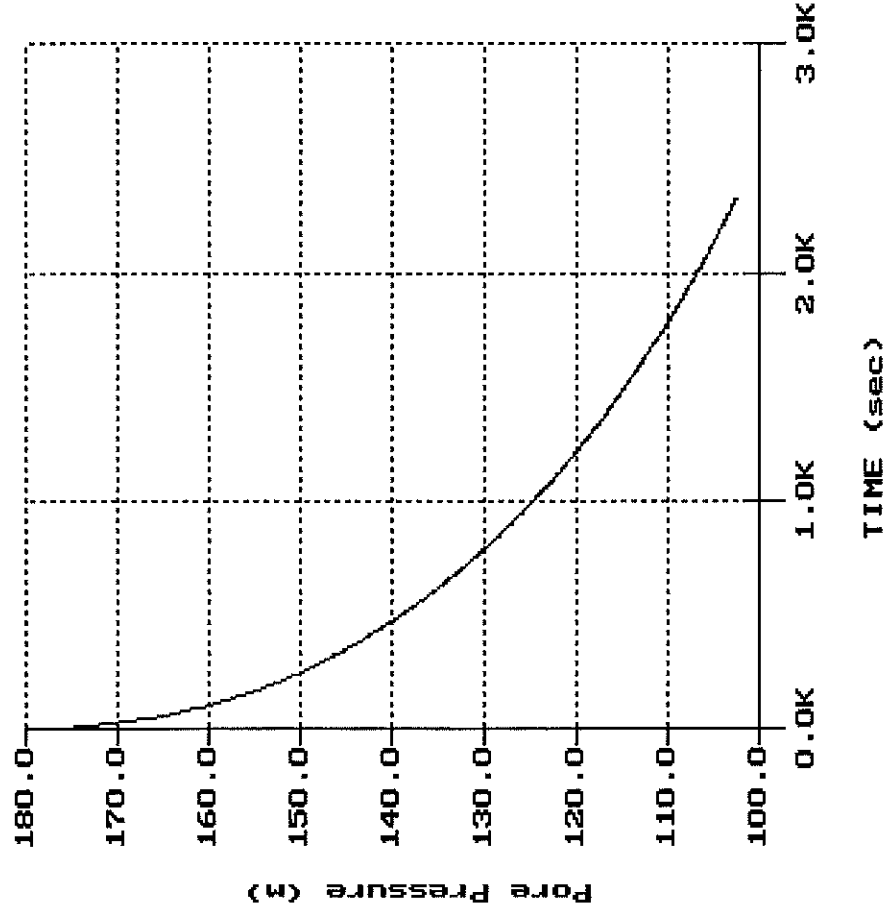
# Thurber

Hole: CPT04-02N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:18:04 11:22

File: 166C02N.PPD  
Depth (m): 34.15  
Duration (ft): 112.04  
U-min: 102.59 2320.0s  
U-max: 178.64 0.0s

## PORE PRESSURE DISSIPATION RECORD

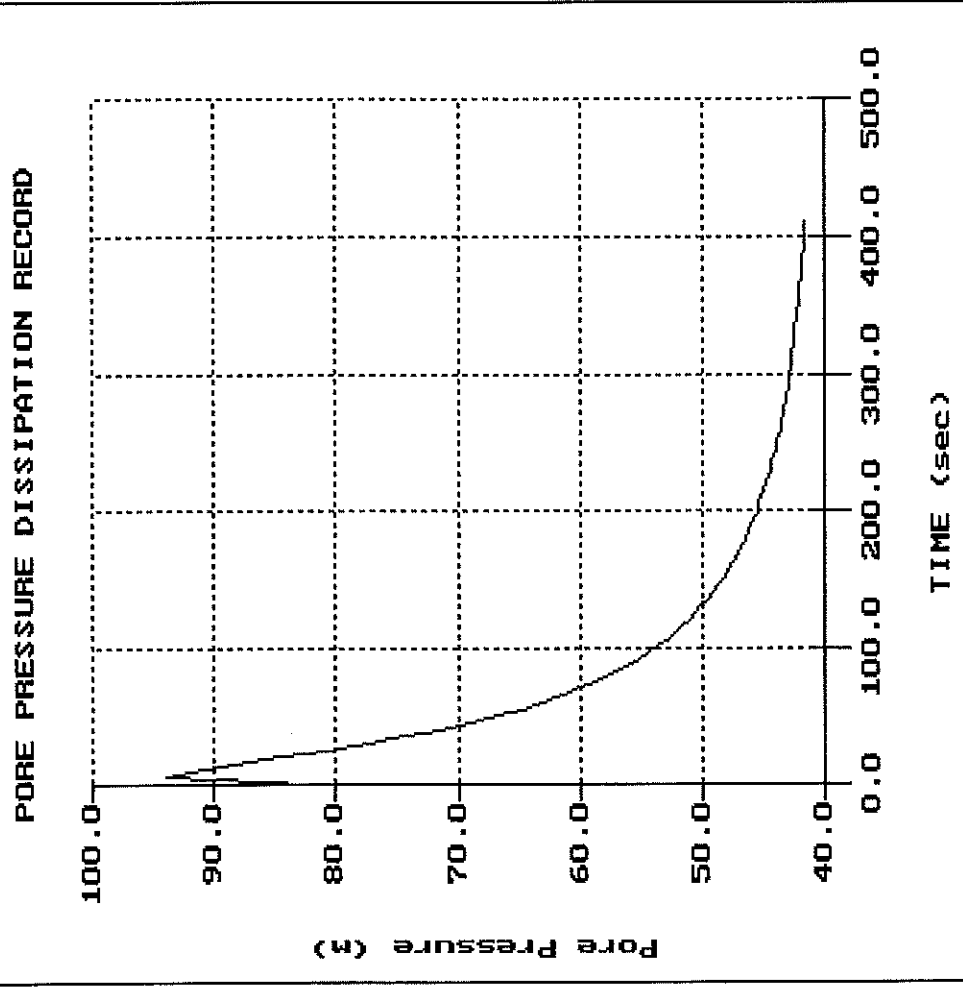


# Thurber

Hole: CPT04-02N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:18:04 11:22

File: 166C02N.PPD  
Depth (m): 43.12  
Depth (ft): 141.47  
Duration: 410.0s  
U-min: 41.57 410.0s  
U-max: 93.96 5.0s

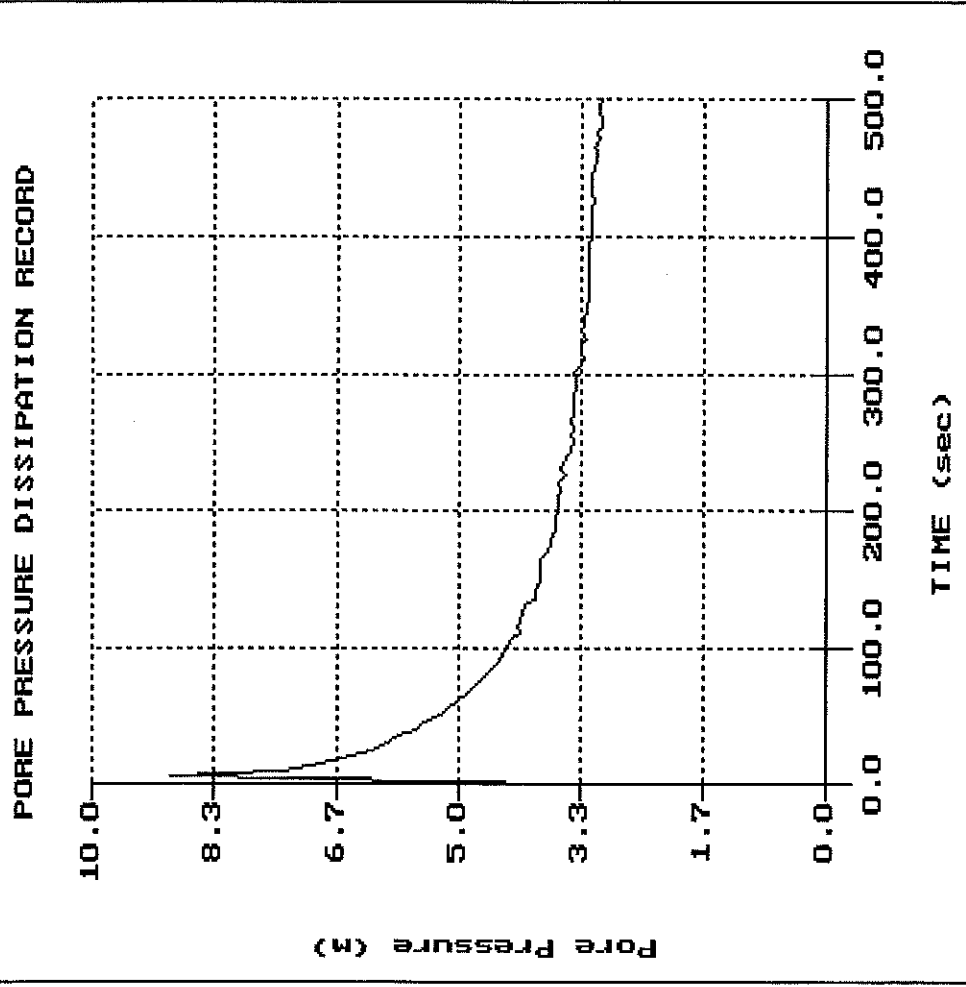


Thurber

Hole: CPT04-02S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:18:04 08:09

File: 166C02S.PPD  
Depth (m): 7.18  
Duration: 23.56  
U-min: 500.0s  
U-max: 3.05 500.0s  
U-max: 8.94 5.0s

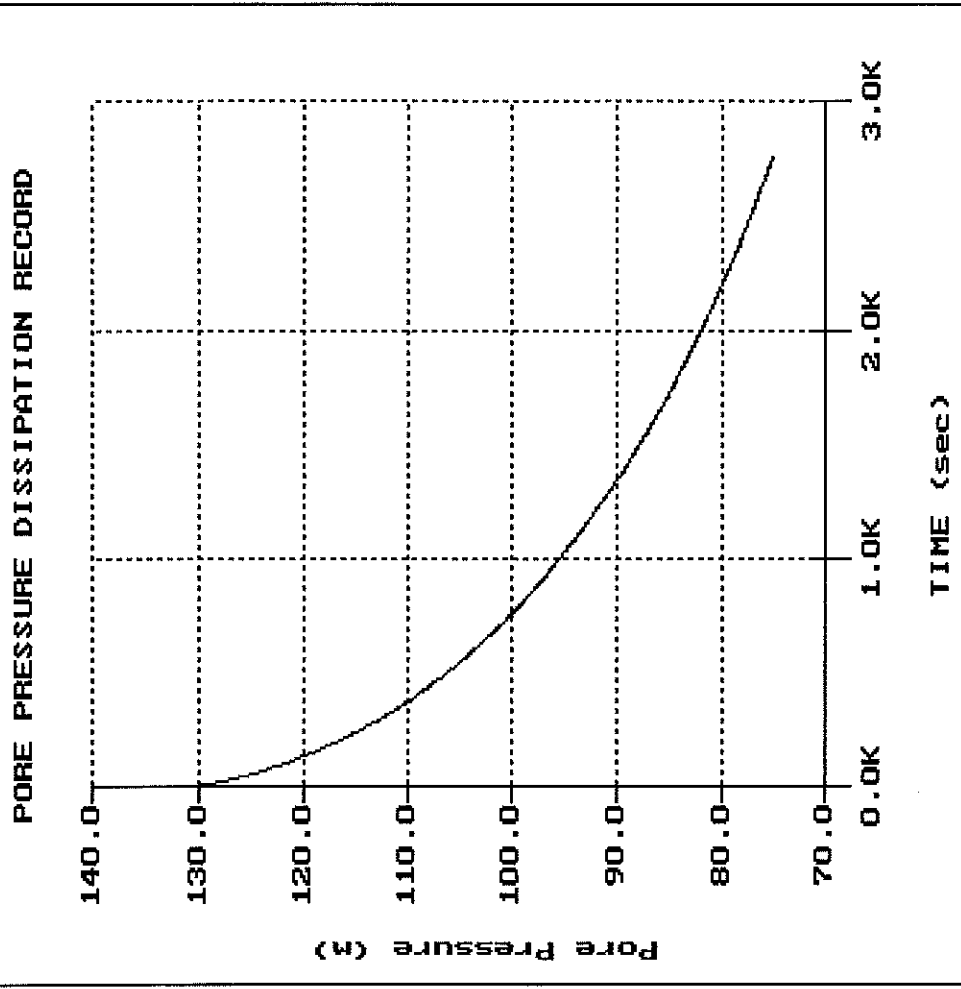


# Thurber

Hole: CPT04-02S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:18:04 08:09

File: 166C02S.PPD  
Depth (m): 28.20  
Depth (ft): 92.52  
Duration: 2750.0s  
U-min: 75.09 2750.0s  
U-max: 131.87 0.0s



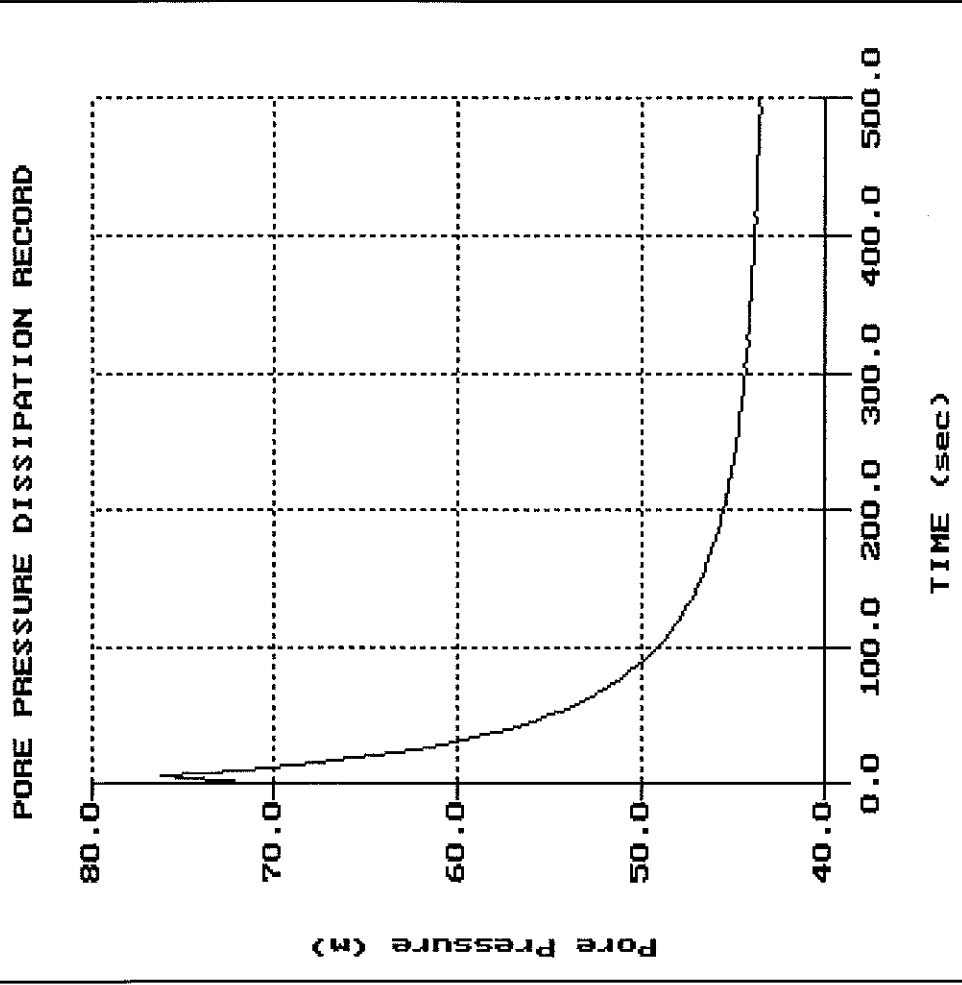


Thurber

Hole: CPT04-02S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:18:04 08:09

File: 166C02S.PPD  
Depth (m): 47.12  
Depth (ft): 154.59  
Duration: 500.0s  
U-min: 43.54 490.0s  
U-max: 76.31 5.0s

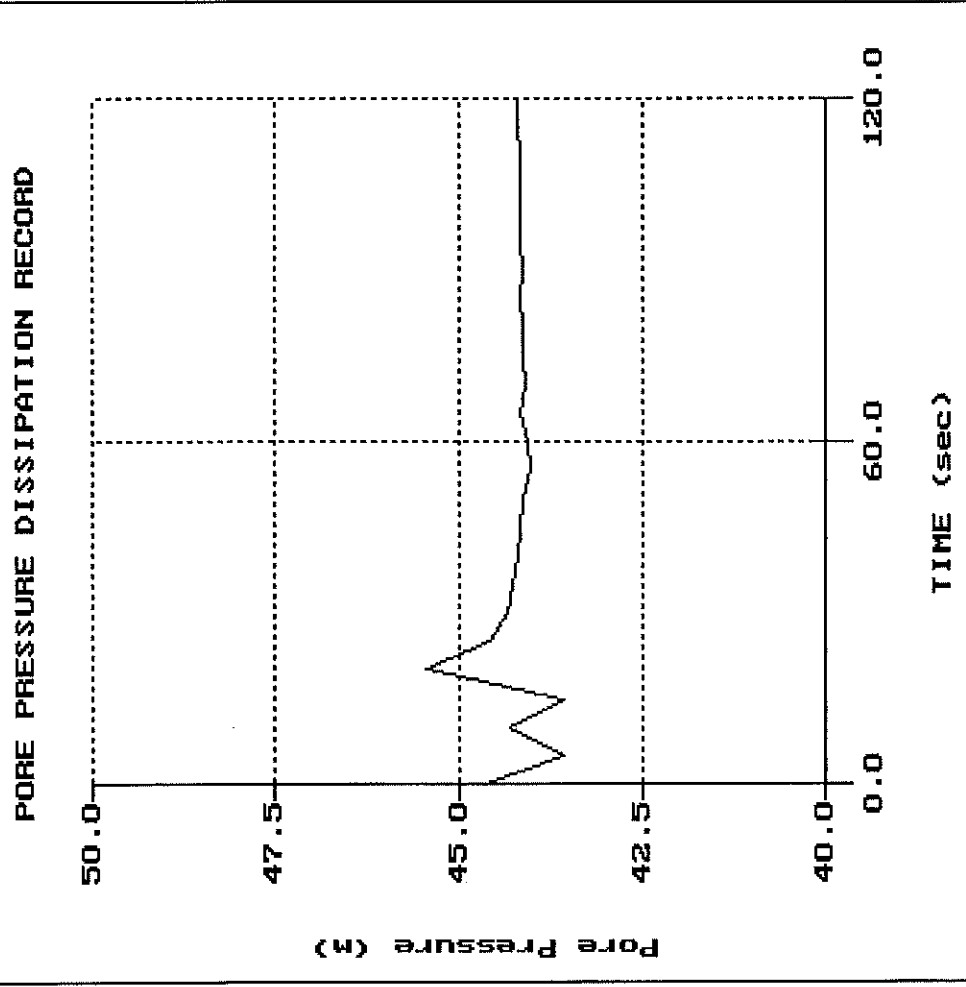


Thurber

Hole: CPT04-02S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:18:04 08:09

File: 166C02S.PPD  
Depth (m): 49.67  
Depth (ft): 162.96  
Duration: 120.0s  
U-min: 43.57 15.0s  
U-max: 45.46 20.0s

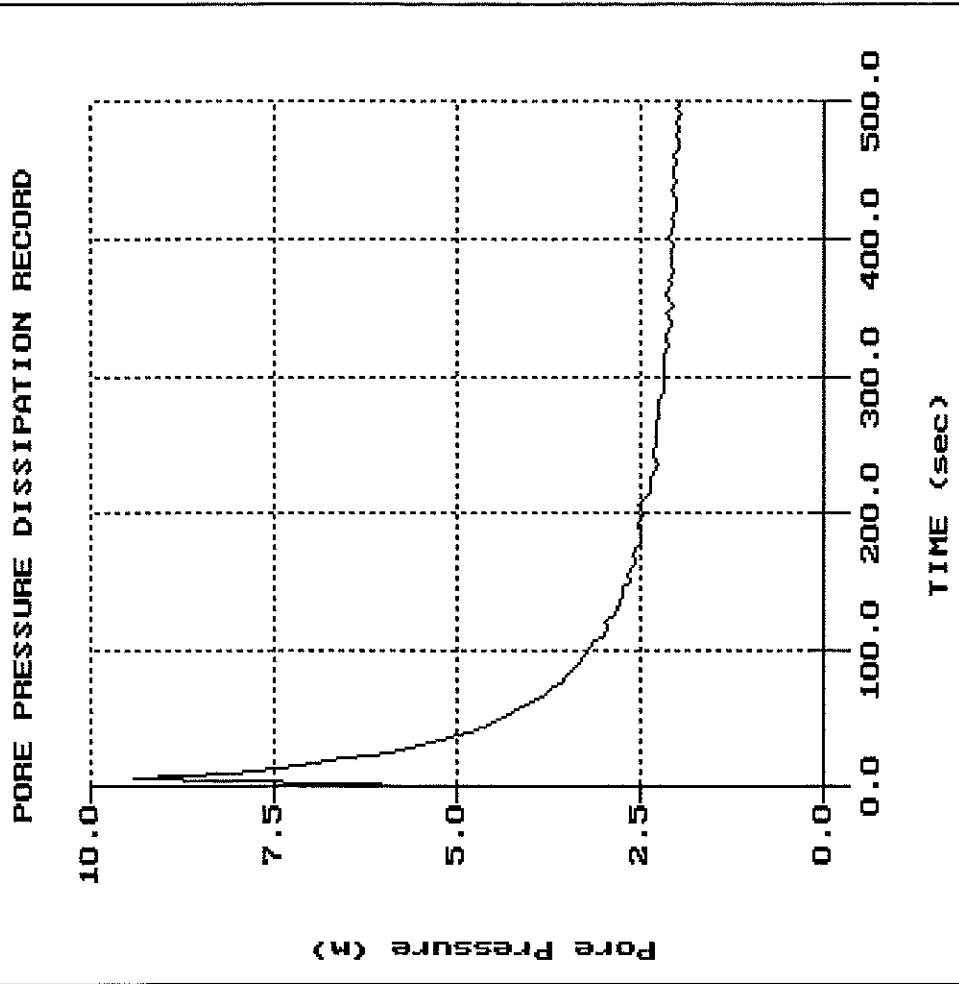


Thurber

Hole: CPT04-03N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:15:04 13:06

File: 166C03N.PPD  
Depth (m): 8.17  
Depth (ft): 26.80  
Duration: 500.0s  
U-min: 1.95 500.0s  
U-max: 9.40 5.0s

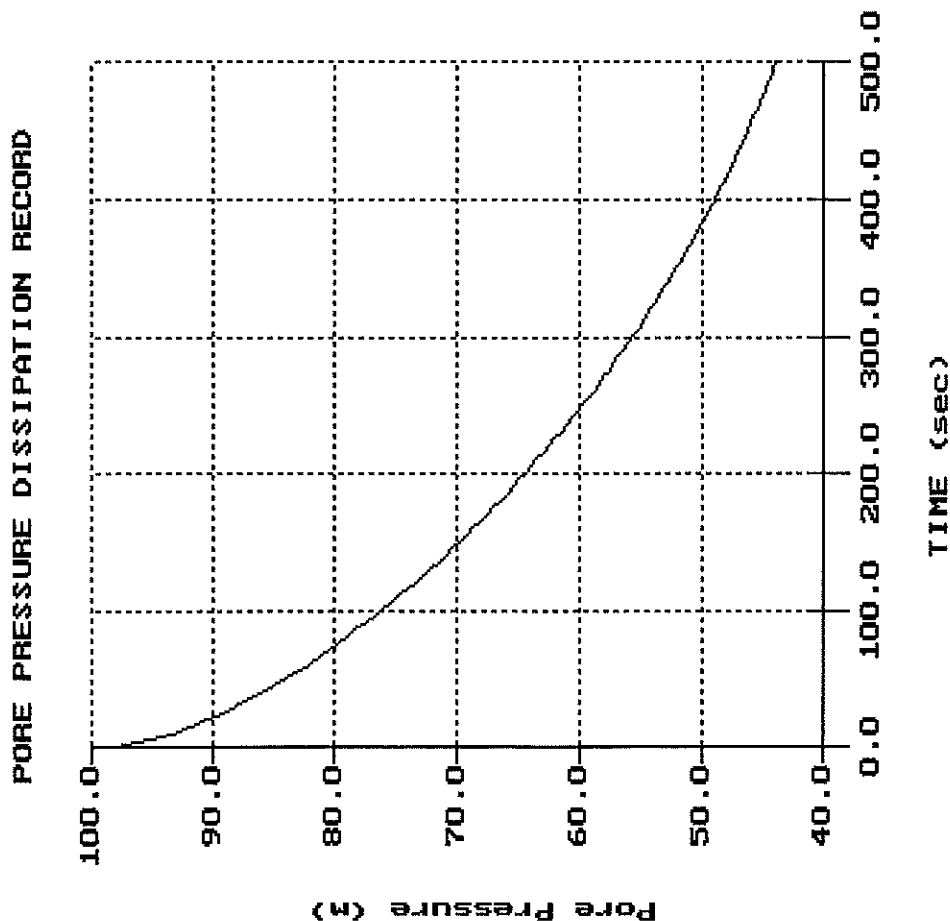


Thurber

Hole: CPT04-03N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:15:04 13:06

File: 166C03N.PPD  
Depth (m): 19.97  
Depth (ft): 65.52  
Duration: 500.0s  
U-min: 43.88 500.0s  
U-max: 98.11 0.0s

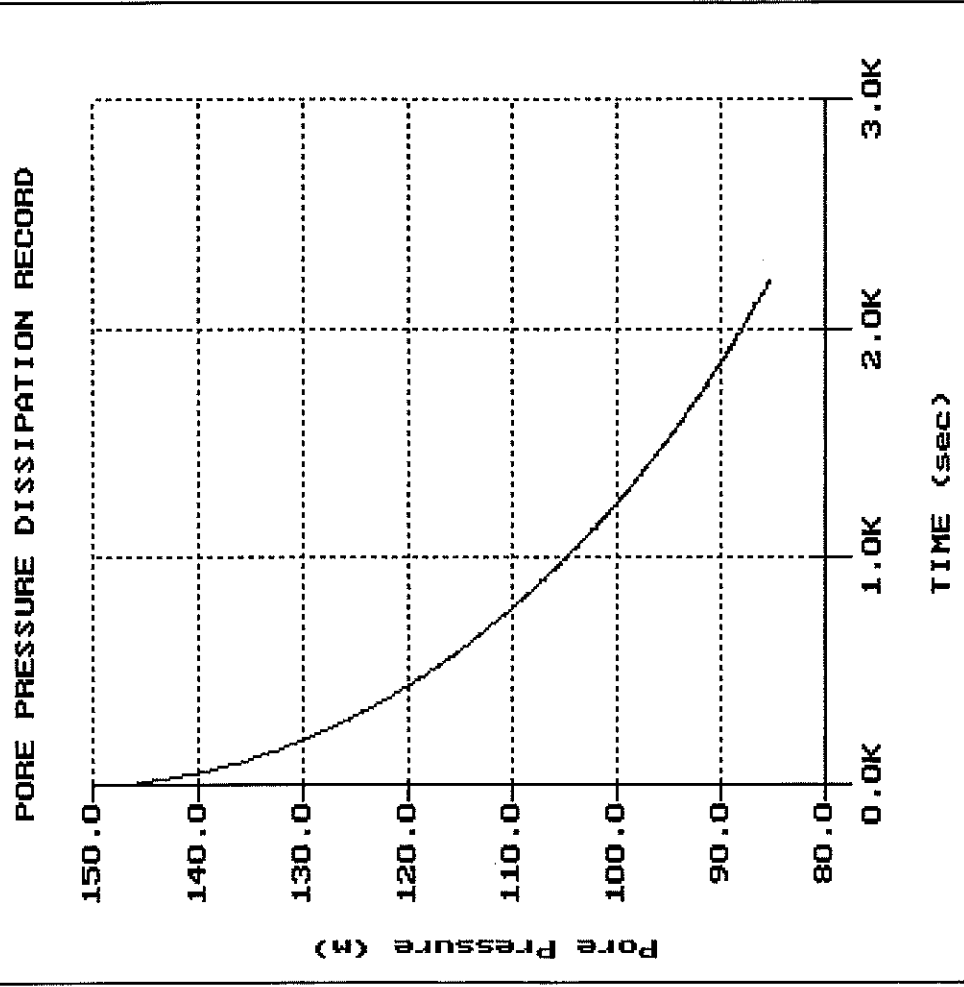


# Thurber

Hole: CPT04-03N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:15:04 13:06

File: 166C03N.PPD  
Depth (m): 33.97  
Duration: 111.45  
U-min: 85.39 2200.0s  
U-max: 148.72 0.0s

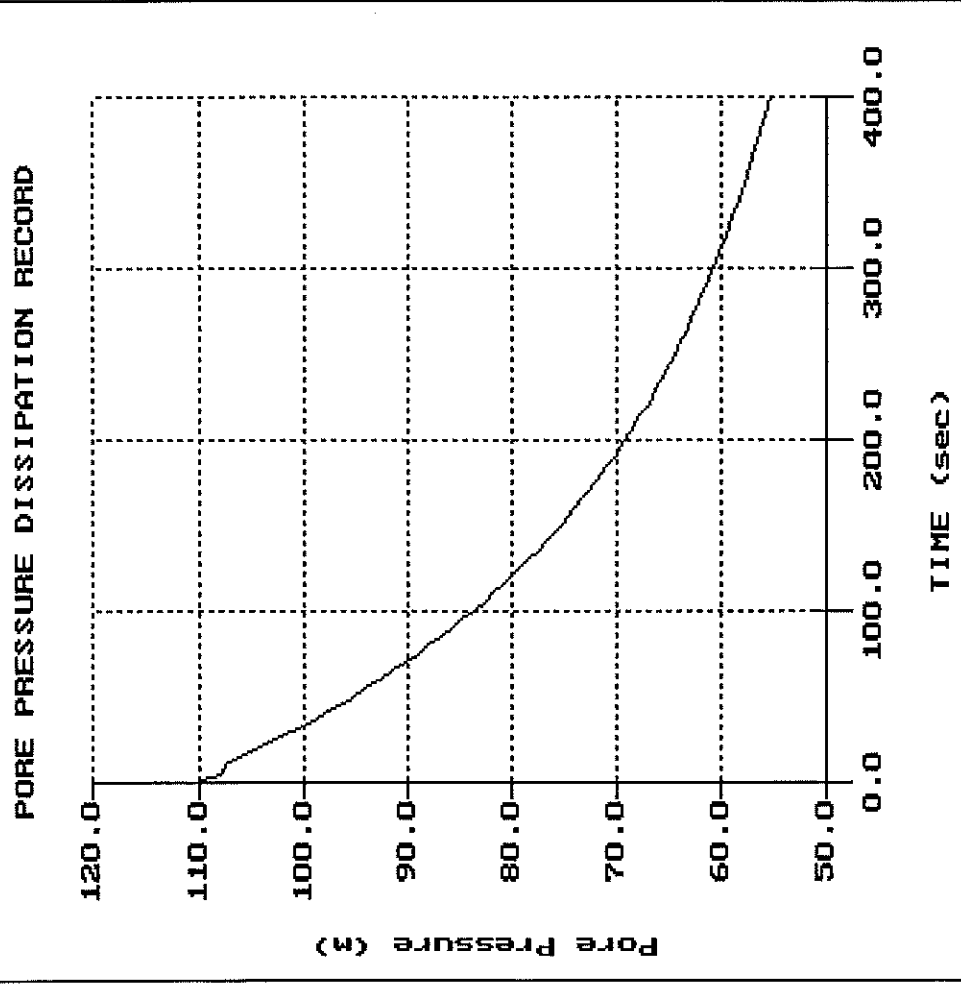


Thurber

Hole: CPT04-03N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:15:04 13:06

File: 166C03N.PPD  
Depth (m): 42.90  
Duration (ft): 140.75  
U-min: 55.41 400.0s  
U-max: 110.27 0.0s

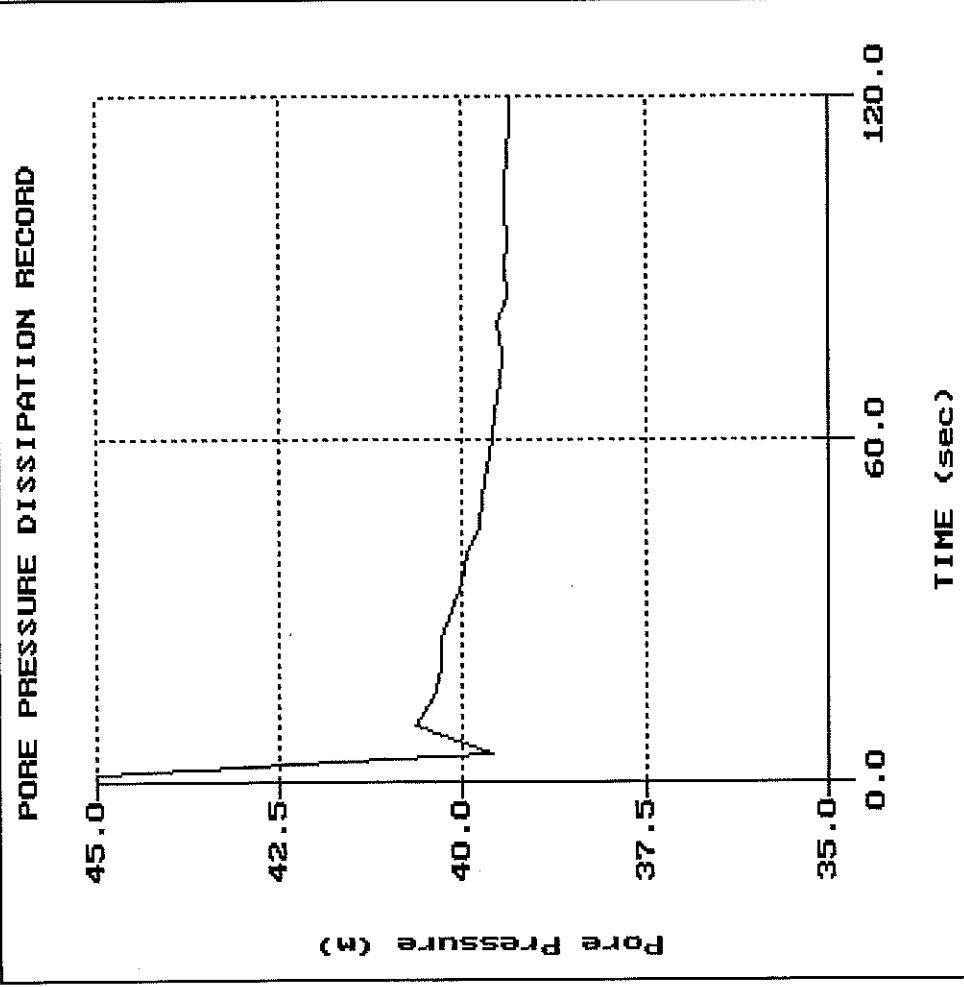


# Thurber

Hole: CPT04-03N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:15:04 13:06

File: 166C03N.PPD  
Depth (m): 46.35  
Duration (ft): 152.07  
U-min: 39.35 120.0s  
U-max: 47.69 0.0s

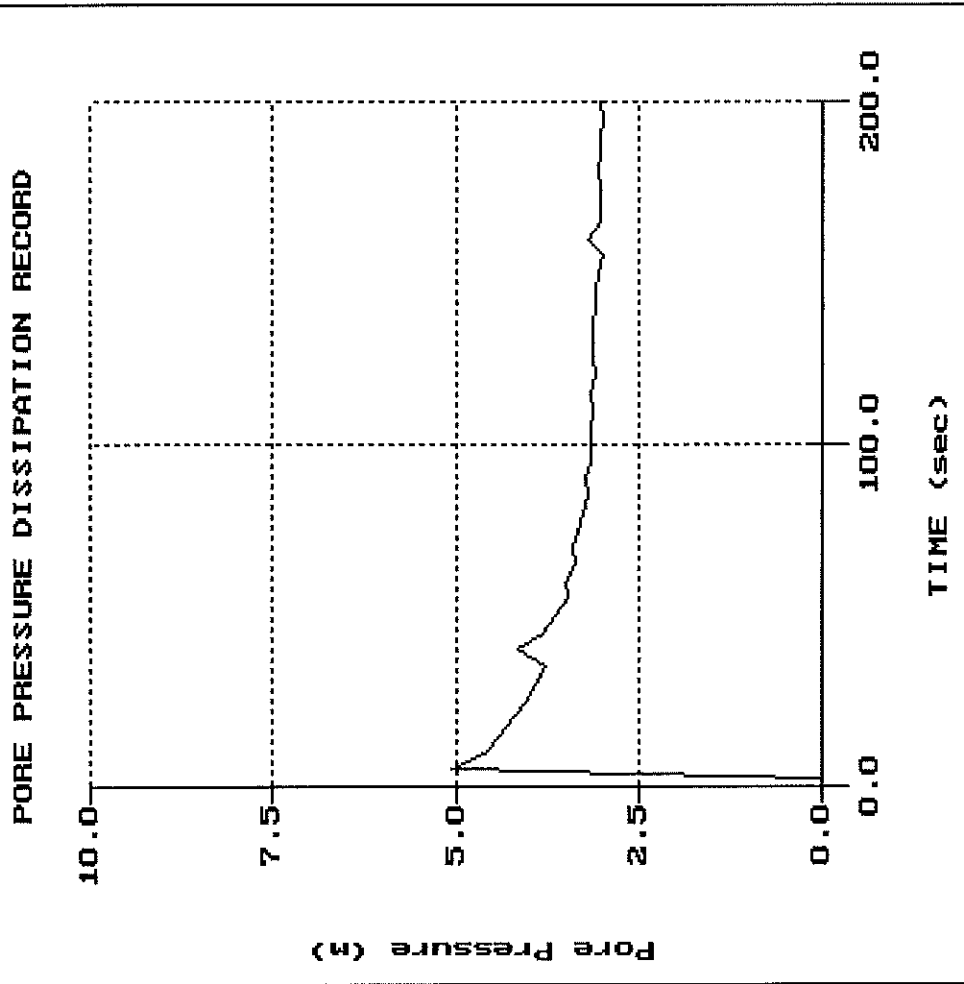


# Thurber

Hole: CPT04-03S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:15:04 07:59

File: 166C03S.PPD  
Depth (m): 10.30  
Depth (ft): 33.79  
Duration: 200.0s  
U-min: -5.19 0.0s  
U-max: 5.08 5.0s



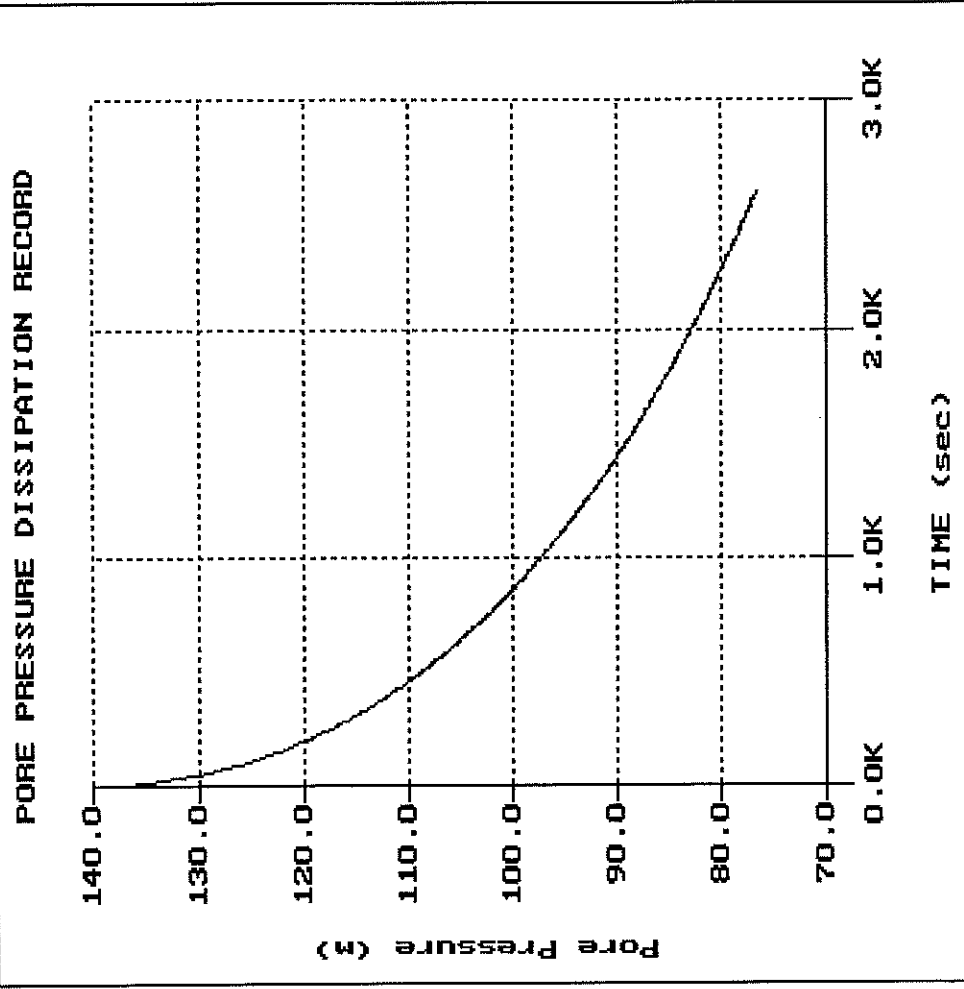


# Thurber

Hole: CPT04-03\$  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:15:04 07:59

File: 166C03\$.PPD  
Depth (m): 27.93  
Depth (ft): 91.63  
Duration: 2600.0s  
U-min: 76.60 2600.0s  
U-max: 139.38 0.0s

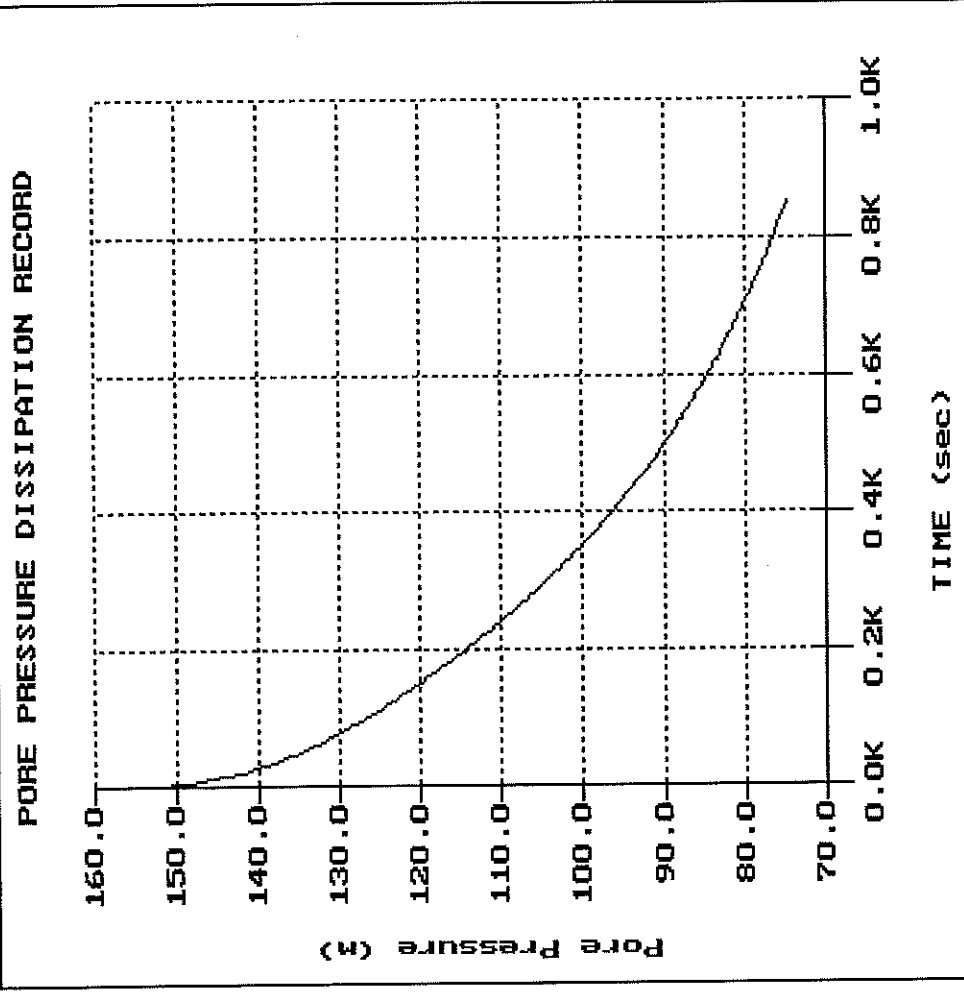


# Thurber

Hole: CPT04-03S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:15:04 07:59

File: 166C03S.PPD  
Depth (m): 40.25  
Duration: 132.05  
U-min: 74.84 850.0s  
U-max: 153.24 0.0s

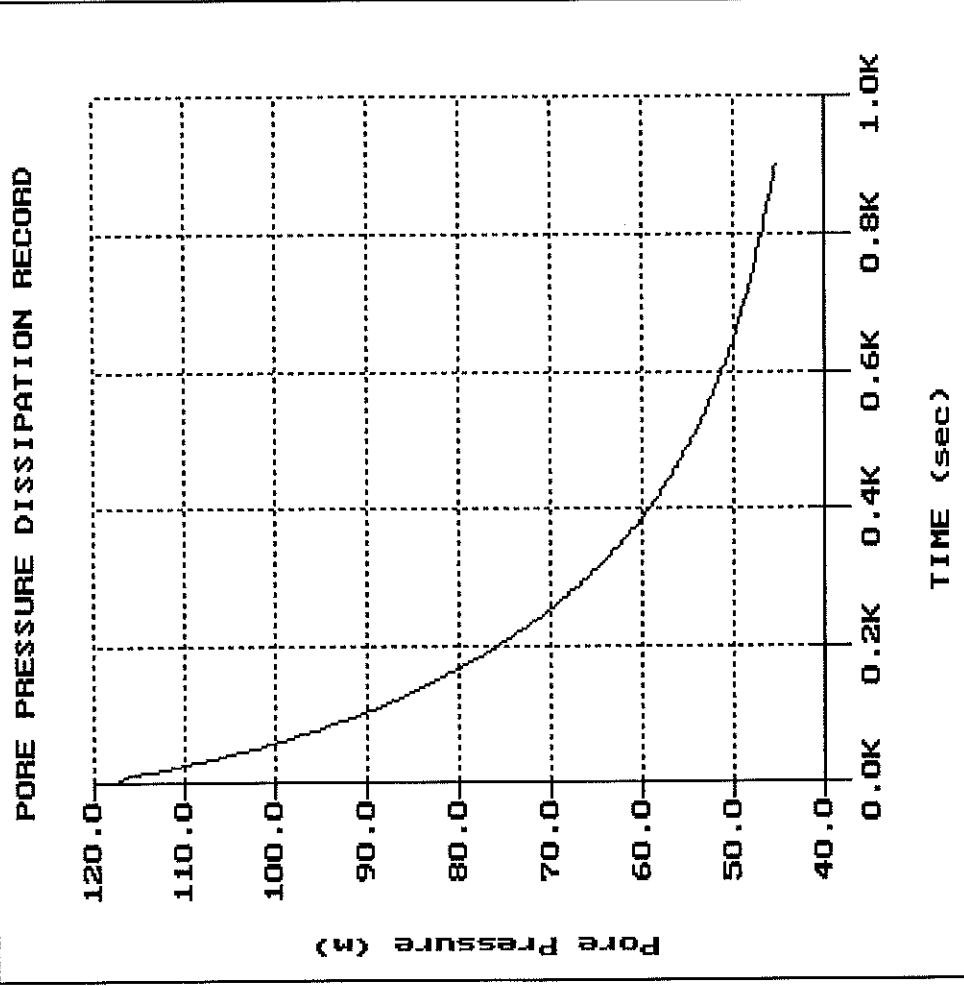


# Thurber

Hole: CPT04-03S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:15:04 07:59

File: 166C03S.PPD  
Depth (m): 41.25  
Duration: 900.0s  
U-min: 45.41 900.0s  
U-max: 117.19 5.0s



# Thurber

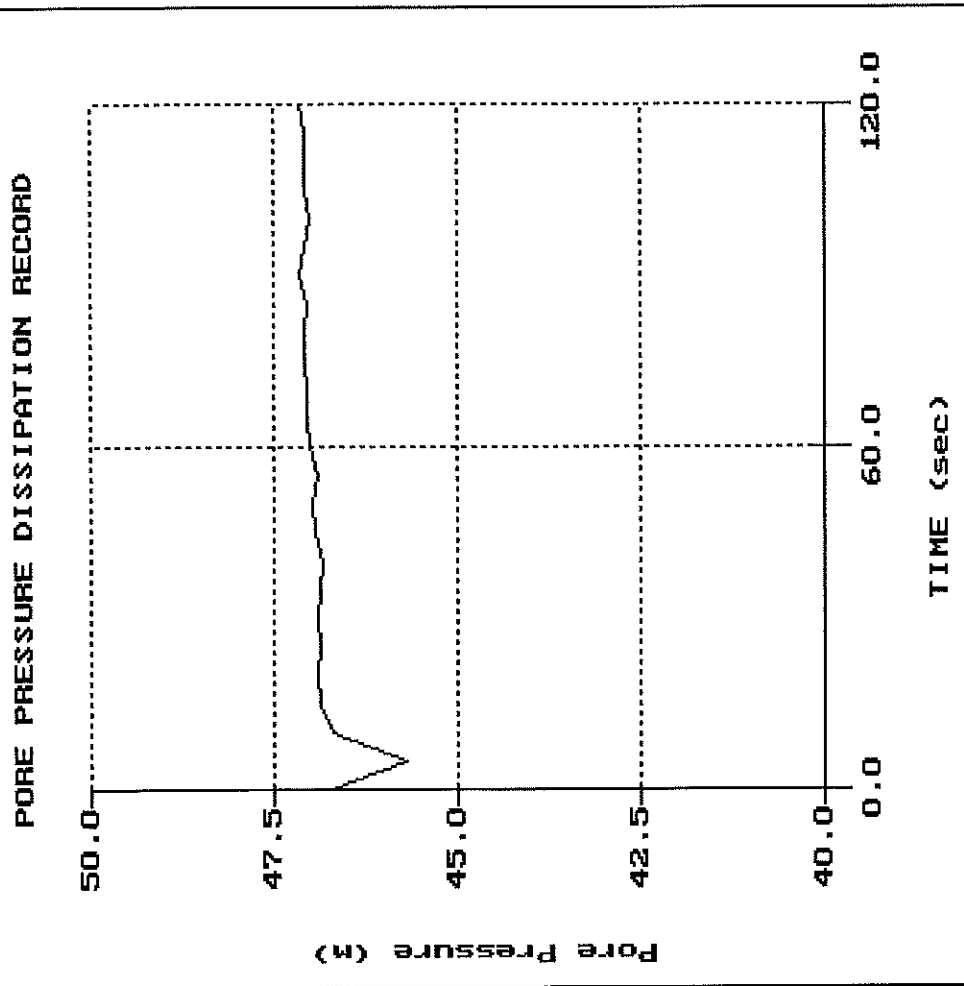
Hole: CPT04-03S

Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113

Date: 05:15:04 07:59

File: 166C03S.PPD  
Depth (m): 52.75  
Depth (ft): 173.06  
Duration: 160.0s  
U-min: 45.71 5.0s  
U-max: 47.22 155.0s

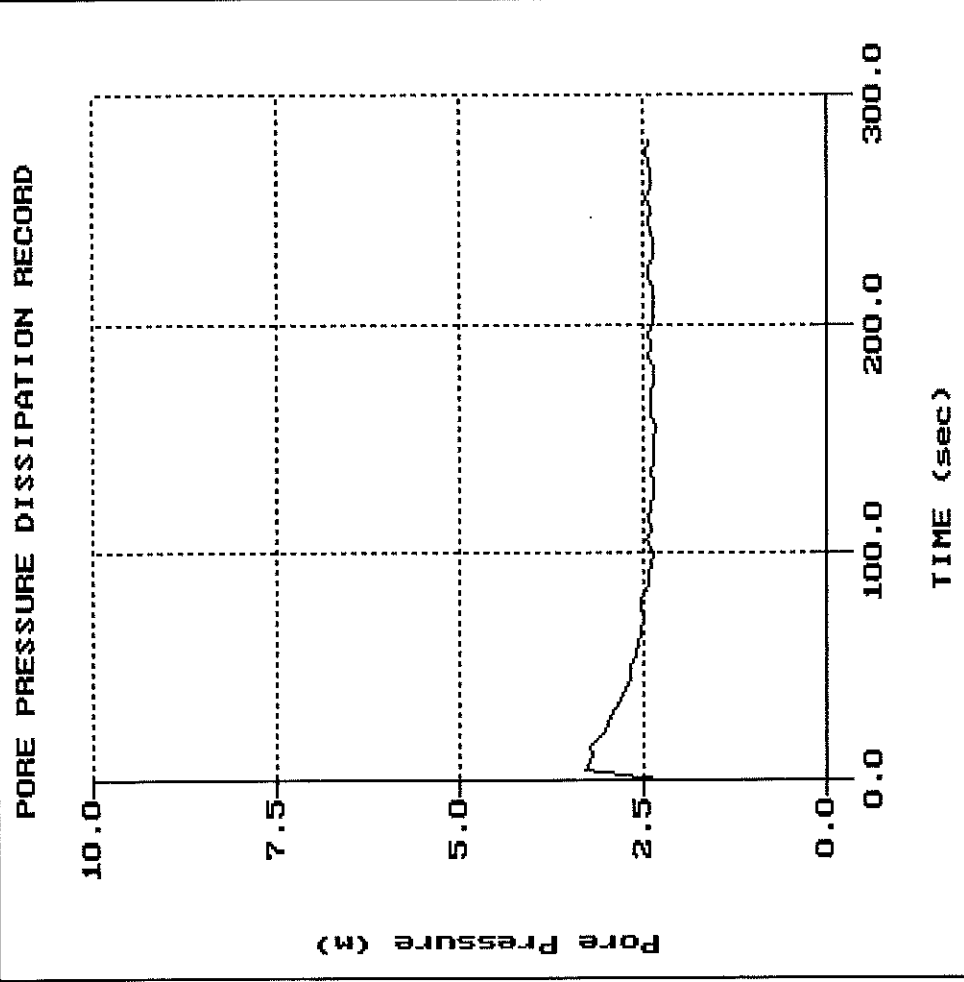


Thurber

Hole: CPT04-04N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:18:04 14:36

File: 166C04N.PPD  
Depth (m): 8.28  
Depth (ft): 27.17  
Duration: 280.0s  
U-min: 2.26 0.0s  
U-max: 3.30 5.0s

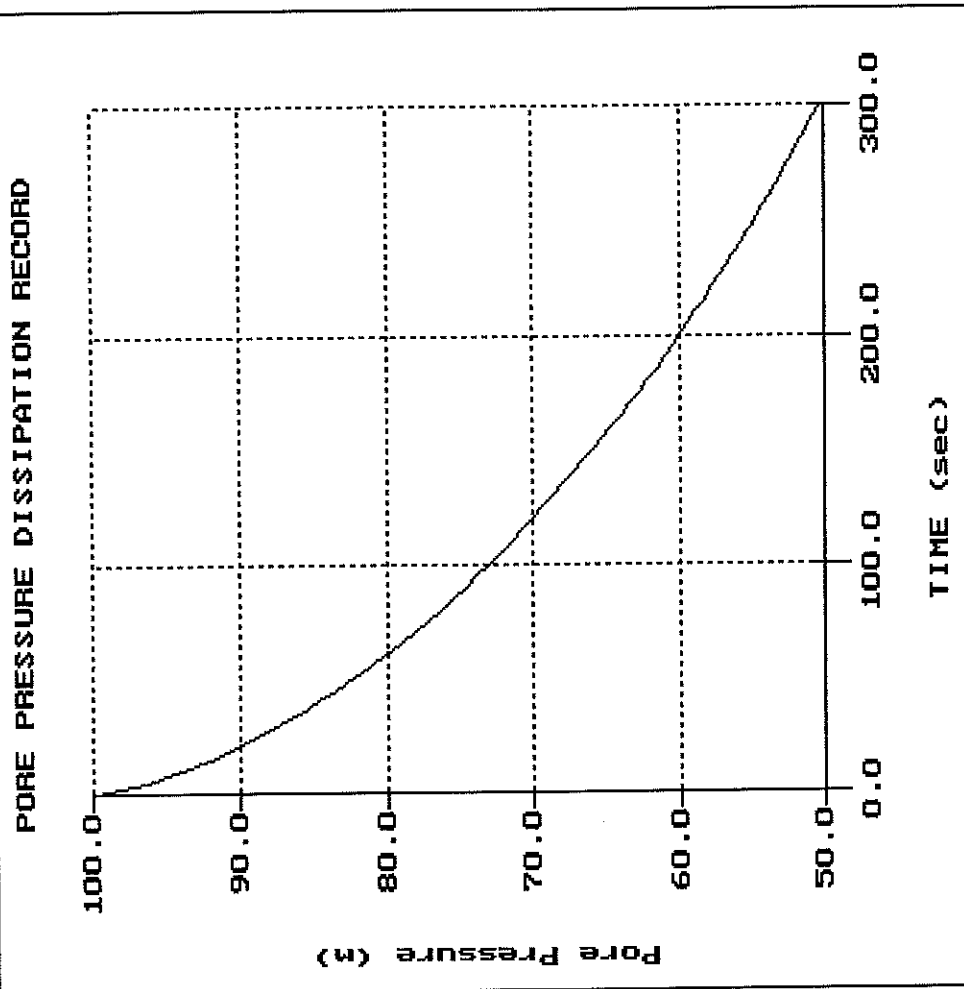


# Thurber

Hole: CPT04-04N  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:18:04 14:36

File: 166C04N.PPD  
Depth (m): 18.27  
Duration: 300.0s  
U-min: 50.33 300.0s  
U-max: 99.80 0.0s

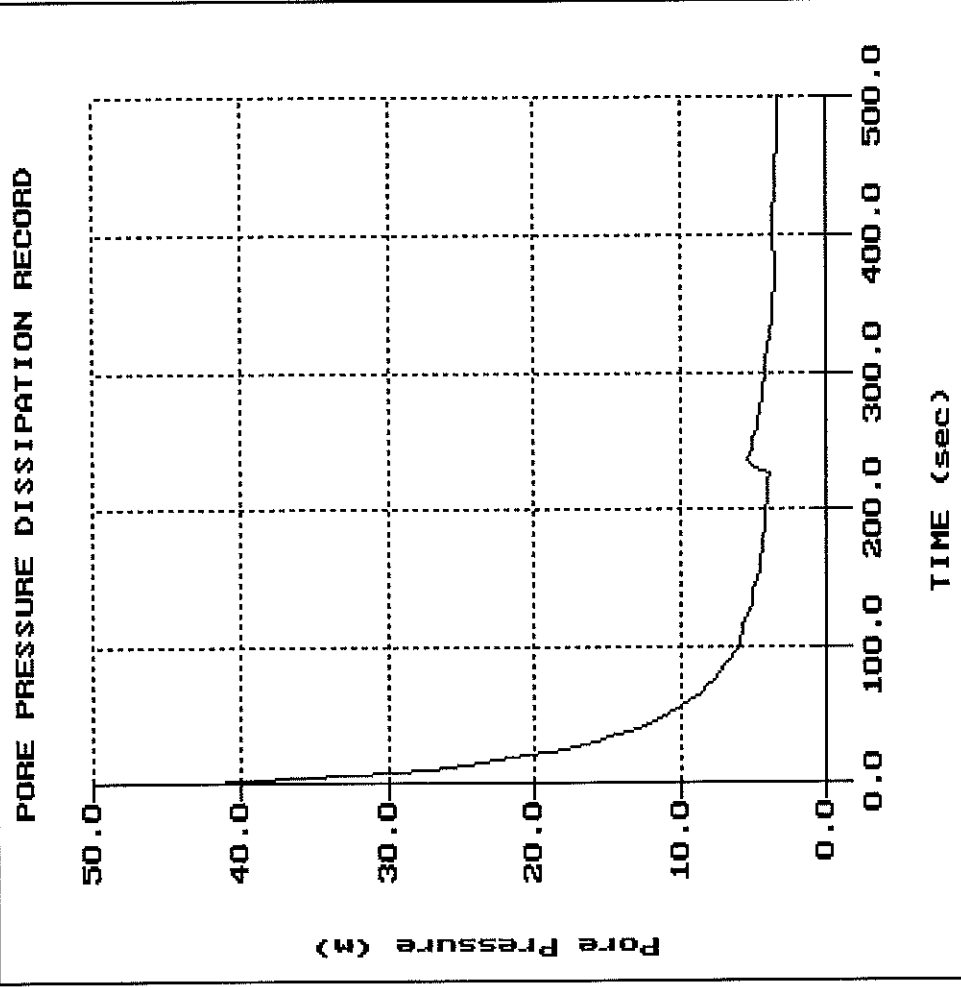


# Thurber

Hole: CPT04-04\$  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:16:04 07:25

File: 166C04\$.PPD  
Depth (m): 7.05  
Duration (ft): 23.13  
U-min: 500.0s  
U-max: 3.26 480.0s  
U-max: 42.85 0.0s



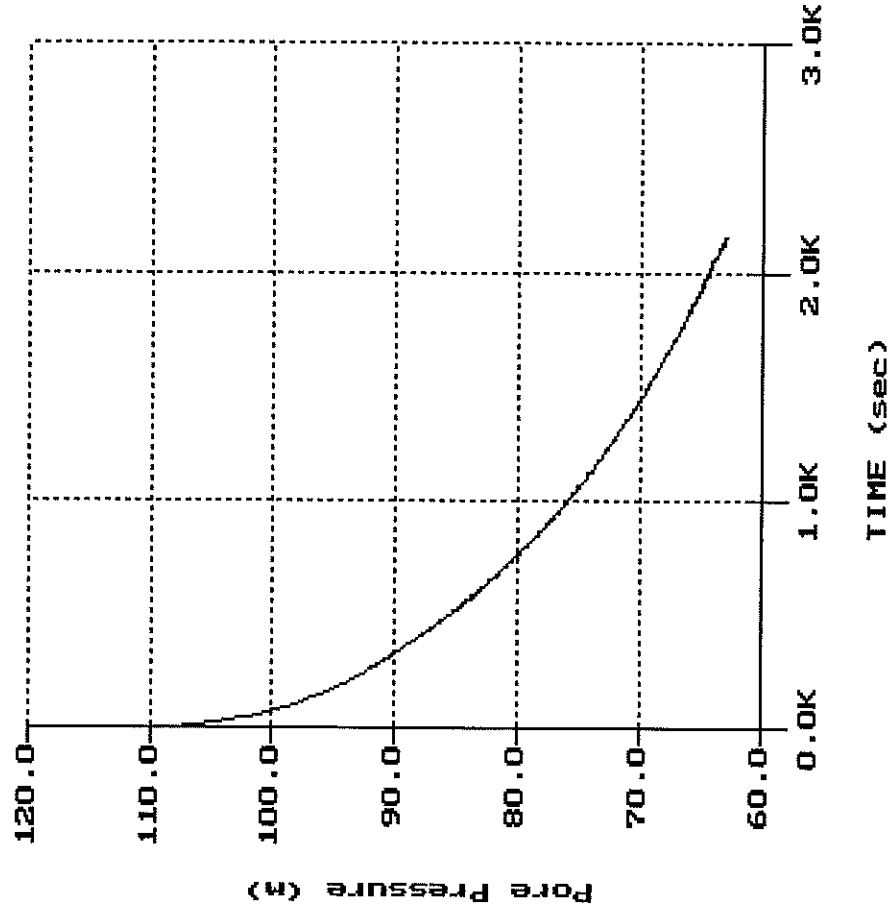
# Thurber

Hole: CPT04-04S  
Location: HWY69 4L ESTAIRE

Cone: 20 Ton St 113  
Date: 05:16:04 07:25

File: 166C04S.PPD  
Depth (m): 24.05  
Depth (ft): 78.90  
Duration: 2150.0s  
U-min: 63.01 2150.0s  
U-max: 110.85 0.0s

## PORE PRESSURE DISSIPATION RECORD





**Appendix E**  
**ConeTec Inc.'s Seismic Analysis Report**



# ConeTec Investigations Ltd.

Geotechnical, Environmental & Marine Site Investigations

9113 Shaughnessy St., Vancouver, BC V6P 6R9 • Tel: (604) 327-4311 • Fax: (604) 327-4066 • Email: [insitu@conetec.com](mailto:insitu@conetec.com)

June 15, 2004

RECEIVED JUN 16 2004

Mr. Paulo Branko  
Thurber Engineering Ltd.  
Suite 103, 2010 Winston Park Drive  
Oakville, Ontario L6H 5R7

Dear Paulo,

RE: Highway 69-4 Lane - Estaire, ON

For demonstration purposes we have used the CPT and Shear Wave (Vs) Velocity data from SCPT04-01S to evaluate the following soil characteristics relating to liquefaction considerations at the referenced site:

- cyclic liquefaction potential using the CPTU
- static liquefaction potential using the CPTU
- settlement potential due to cyclic liquefaction using the CPTU
- lateral spread potential due to cyclic liquefaction using the CPTU
- cyclic liquefaction potential using Vs
- static liquefaction potential using Vs
- sand compressibility using CPTU and Vs

A detailed discussion on the application of the CPT for liquefaction evaluation is presented in a letter from Dr. Peter Robertson dated June 1, 2004 and in his recent publication "Evaluating Soil Liquefaction Using the CPT", Robertson 2004.

A detailed discussion on the application of shear wave velocity for liquefaction evaluation is presented by Youd, et.al, 2001 pp 303 & 304.

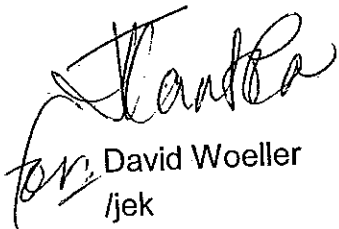
A detailed discussion on the "Estimation of Sand Compressibility from Seismic CPT" is presented by Robertson et.al., 1996.

Copies of these reference papers are appended to this data report.

In summary, the demonstration of seismic cone penetration test technology at the Highway 69 site has demonstrated the following with respect to liquefaction:

1. Based on both penetration resistance and shear wave velocity cyclic liquefaction will not occur for the design earthquakes and hence vertical settlements and lateral displacements are insignificant.
2. Some sand layers at the site are very loose and brittle and can undergo significant strain softening in simple shear loading.
3. Comparison of penetration resistance and shear wave velocity indicate that the sands at the site are compressible.

Sincerely,

  
for David Woeller  
/jek

Enclosure

ref:04-166

## Appendix A

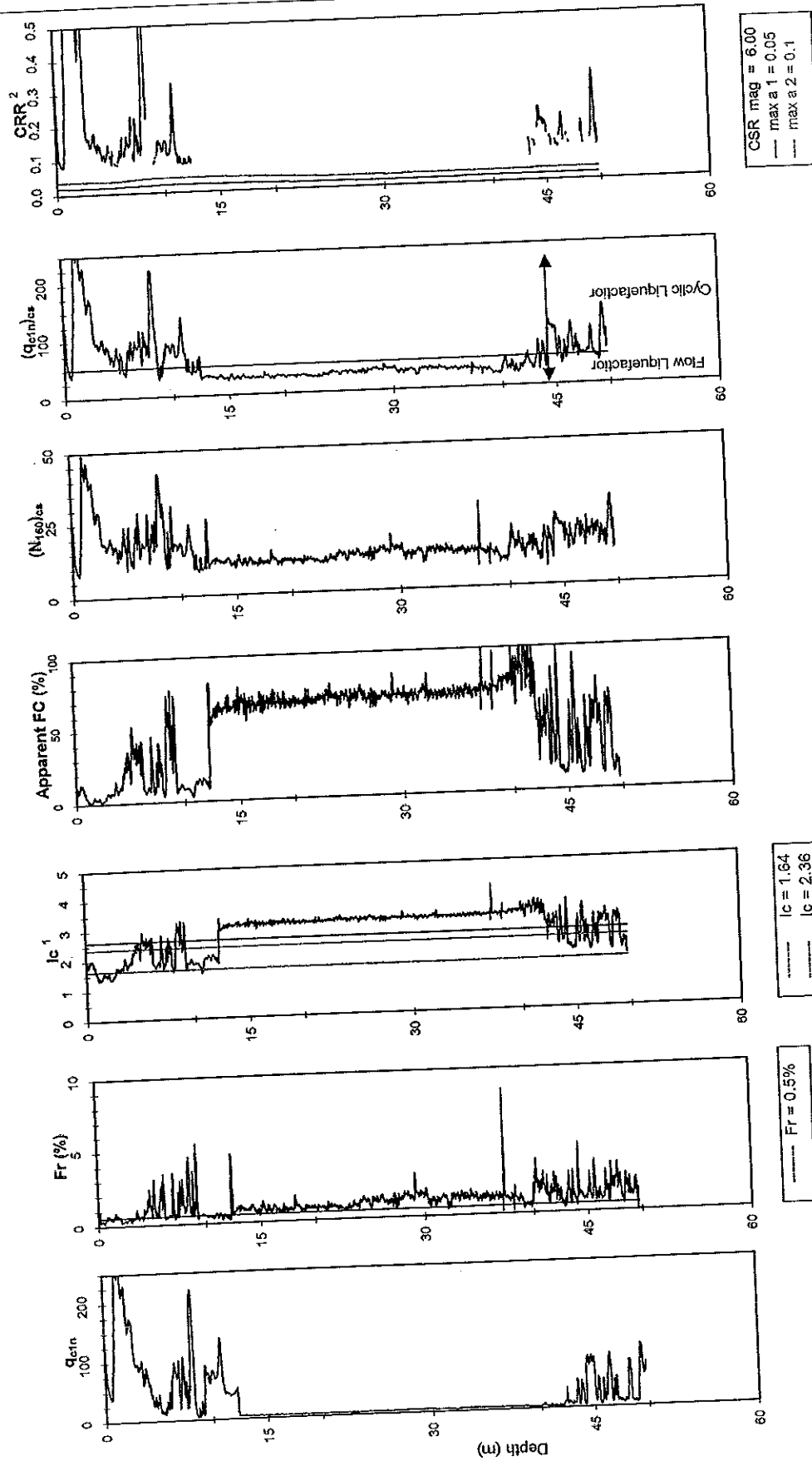
# Application of the CPT for Estimating Liquefaction Parameters



# APPLICATION OF THE INTEGRATED CPT METHOD FOR ESTIMATING LIQUEFACTION PARAMETERS

(Robertson, Wride 1998; Zhang, Robertson, Brachman 2002)

CPT: SCPT04-01S Location: HWY69 4L ESTAIR WT: 5.70m Date: 05/17/04 Hole Depth: 49.70m Plots(1)



6/14/2004 13:26

<sup>1</sup>  $I_c$  greater than 2.6 indicates clayey silt, silty clay, or clay material that is likely not liquefiable. This conclusion should, however, be verified by samples.

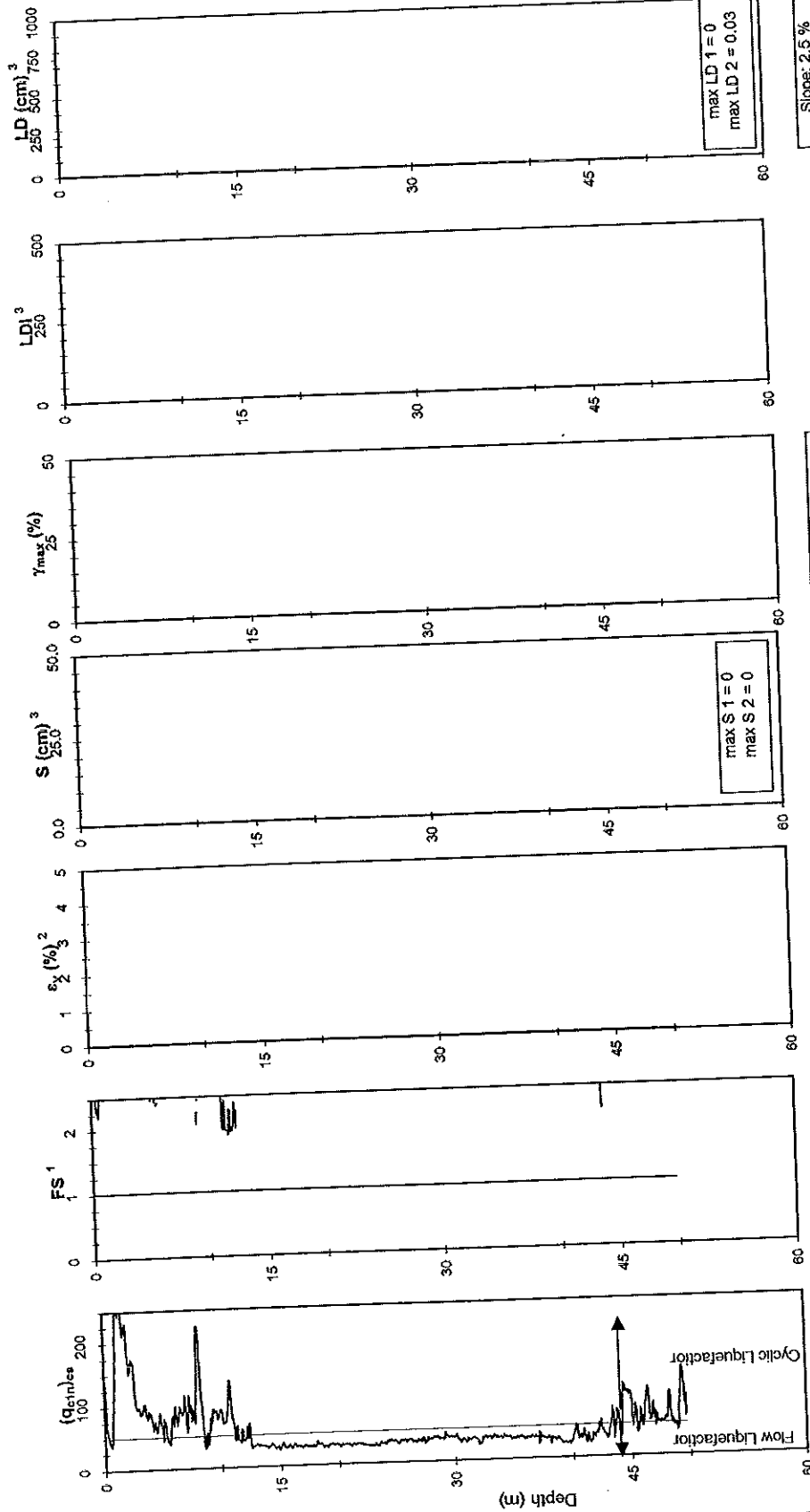
<sup>2</sup> Empirical data indicate no observed liquefaction below 20m (65ft).



# APPLICATION OF THE INTEGRATED CPT METHOD FOR ESTIMATING LIQUEFACTION PARAMETERS

(Robertson, Wride 1998; Zhang, Robertson, Brachman 2002)

CPT: SCPT04-01S Location: HWY69 4L ESTAIR WT: 5.70m Date: 05/17/04 Hole Depth: 49.70m Plots(2)



- 1 Empirical data indicate no observed liquefaction below 20m (65ft).
- 2 Volumetric strain may be overpredicted in interface zones between soil types and in sand layers less than 1m (3ft) thick.
- 3 No liquefaction assumed below a depth of 20m (65ft).

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June 1, 2004

ConeTec Investigations Ltd.

Attention: David Woeller

**Re: Application of CPT for Liquefaction Evaluation**

This letter provides some background regarding the potential application of the Cone Penetration Test (CPT) for the evaluation of liquefaction potential for sub-soils around Ontario highway bridges. SCPT04-01S is used to illustrate the application.

**Liquefaction**

Given the greater reliability and repeatability of the CPT over the Standard Penetration Test (SPT), liquefaction analyses can be carried out using the method proposed by Robertson and Wride (1998) based on the CPT results.

Prediction of soil liquefaction is an imprecise art due to the complex nature of earthquake loading, soil response and the variable factors that influence soil liquefaction. The method by Robertson and Wride (1998) attempts to capture many of these complex factors in a semi-empirical manner. During cyclic loading from a major earthquake, pore pressures can accumulate in sandy soils and effective stresses decrease resulting cyclic softening (liquefaction). Decreases in effective stresses in sandy soils results in cyclic shear strains during the earthquake. If there is sloping ground or level ground with an adjacent 'free-face', or level ground with a large one-sided loading, lateral spreading can occur. After earthquake loading, excess pore pressures dissipate resulting in volumetric strains that accumulate in the form of vertical settlements. Volumetric strains can also occur in sandy soils above the water table due to the cyclic loading. The resulting settlements and displacements occur after the earthquake and are referred to as post-earthquake settlements and displacements. The time for total post-earthquake settlements to occur depends on the length of drainage paths from the sandy layers in which the volumetric strains occur. Typically they can occur during and shortly after the earthquake event.

Liquefaction susceptibility is influenced by soil compositional characteristics that influence volume change due to cyclic loading. These characteristics include particle

shape, size, gradation, and plasticity. Typically the potential for liquefaction is limited to the more sandy layers.

Liquefaction analyses can be carried out on all CPT soundings using the method by Robertson and Wride (1998) (see attached profiles). Each plot shows profiles of normalized cone penetration resistance ( $q_{c1n}$ ), friction ratio, ( $Fr$ ), soil behavior type index ( $I_c$ ), calculated apparent fines content (FC) in percent, calculated equivalent normalized SPT ( $(N_{160})_{cs}$ ), calculated clean sand equivalent normalized cone resistance ( $(q_{c1n})_{cs}$ ), and calculated cyclic resistance ratio (CRR). The groundwater level at the time of the CPT sounding has been used to normalize the CPT results and to calculate the CRR. The calculated CRR is compared to the simplified estimate of the cyclic stress ratio (CSR) from the design earthquake to evaluate if liquefaction is likely. Cyclic stress ratios were estimated for design earthquakes with maximum surface accelerations of 0.05g and 0.10g and  $M_w=6.0$ . Small changes in groundwater level and surface accelerations have a minor effect on the calculated cyclic stress ratio from the earthquake. Based on the method by Robertson and Wride (1998), soils with a CPT soil behavior index,  $I_c > 2.6$  typically behave more like a clay and are likely non-liquefiable.

The method by Robertson and Wride (1998) was designed as a simplified approach to estimate soil liquefaction based on CPT results. Selected samples are required to clarify interpretation in fine-grained soils. Profiles produced using this method require engineering judgment to aid in the final CPT interpretation due to a number of factors that the method cannot directly take into account. Judgment is required to account for the following:

- Depth effects,
- Interface effects, and,
- Thin layer effects.

Field observations have shown that there has been very little evidence of significant soil liquefaction below a depth of about 20m. Hence, it is reasonable to assume that little to no liquefaction will occur below a depth of 20m.

CPT data are collected at very close intervals, typically 2 to 5 cm. When a cone penetrometer is approaching an interface between a softer clay and denser sand, the cone will sense the soil both ahead and behind the interface. Hence, the CPT results can incorrectly indicate a very thin zone of potential liquefaction due to the transition from one soil type to another. This occurs when the cone is penetrating from a softer silt/clay into a sand layer and again when the cone leaves the sand and re-enters the softer silt/clay. The CPT method, because of the high frequency of the data, can incorrectly conservatively predict a very thin zone of liquefaction above and below sand layers due to this interface effect.

When a sand layer is less than about 1m thick, the interface effect results in the cone sensing soft silt/clay below the sand layer before the cone can sense the full cone resistance in the thin sand layer. Hence, the sand layer appears looser than it is due to the



limited layer thickness. This can result in the prediction of potential liquefaction in thin sand layers.

For the example shown, cyclic liquefaction will not occur for design earthquakes with maximum surface accelerations of 0.05g and 0.10g and  $M_w=6.0$ .

### Settlement and Displacement Predictions

Early methods to evaluate soil liquefaction were based solely on Factor of Safety calculated by comparing the CRR of the soil with the design cyclic stress ratio (CSR) for the earthquake. However, field evidence from recent major earthquakes have illustrated that this can be an overly conservative approach. The current approach is to estimate ground deformations after the design earthquake, since ground deformations control structure performance after a design earthquake. Ground deformations can be vertical post-earthquake settlements and lateral spreading due to non-level ground geometry.

Predicting deformations under simple static loading on sandy soils is often difficult due to the natural variability of most sandy deposits. Estimating post-earthquake settlements and displacements can be particularly difficult with large uncertainties. Zhang, Robertson and Brachman (2002 and 2003) suggested an extension to the Robertson and Wride (1998) CPT method to provide estimates of post-earthquake settlements and displacements. Comparisons with case history results have shown that this method can provide reasonable, yet conservative, estimates of post-earthquake settlements and displacements, (Zhang, Robertson and Brachman, 2002 and 2003).

Results of analyses using the Zhang, Robertson and Brachman (2002 and 2003) approach are shown in the attached profiles. Each plot shows profiles of calculated clean sand equivalent normalized cone resistance  $((q_{c1n})_{cs})$ , Factor of Safety against liquefaction (FS), calculated volumetric strain ( $\epsilon_v$ ), calculated cumulated settlement (S), calculated maximum shear strain, lateral displacement index (LDI) and, calculated lateral displacement (LD) based on the ground geometry.

As with the evaluation of liquefaction, engineering judgment is required to aid in the final interpretation of the CPT analyses. Engineering judgment is required to reduce the 'calculated' settlement due to the following:

- Depth effects,
- Interface effects, and,
- Thin layer effects.

As with the evaluation of liquefaction, settlement and displacement estimates need to be adjusted, since the above effects tend to increase 'calculated' deformations.

For the example shown, the post-earthquake settlements for design earthquakes with maximum surface accelerations of 0.05g and 0.10g and  $M_w=6.0$  are estimated to be less than 1cm and the post-earthquake lateral-displacements are estimated to be less than 1cm.

These analyses have been carried out under the current ground and groundwater conditions.

## Flow Liquefaction

When a soil is either very loose or very sensitive and will experience undrained strain softening, there is the potential for instability. The key response parameter required to estimate if a flow slide would occur due to flow liquefaction is the minimum (liquefied) undrained shear strength,  $s_{min}$  ( $s_{LIQ}$ ) following strain softening. Methods have been suggested to estimate the minimum (liquefied) undrained shear strength of clean sands from penetration resistance based on case histories (Seed and Harder, 1990; Stark and Mesri, 1992; Yoshimine et al, 1999; Olsen and Stark, 2002). Recent research has also shown the importance of direction of loading on the minimum undrained shear strength of sands. The minimum undrained shear strength in triaxial compression (TC) loading is higher than that in simple shear (SS), which in turn, is larger than that in triaxial extension (TE). The difference is less as the sand becomes looser. Hence, the appropriate undrained shear strength to be used in a stability analyses will be a function of direction of loading. This has been recognized for some time in clay soils (Bjerrum, 1972). The possibility of instability in undrained shear is also linked to the brittleness or sensitivity of the soil.

Yoshimine et al. (1999) proposed a relationship between normalized cone penetration resistance and the minimum undrained shear strength in simple shear loading for clean sands based on a combination of laboratory testing and case histories. The method suggested by Robertson and Wride (1998), can be used to calculate the equivalent clean sand normalized penetration resistance,  $(q_{c1N})_{cs}$  to estimate the potential for strain softening behavior.

Olsen and Stark (2002) reviewed case histories of flow liquefaction and showed that there was no evidence of flow liquefaction where the mean normalized cone resistance ( $q_{c1N}$ ) was greater than 65, although only two data points (out of 32) exceeded a value of 50. They also showed that when the normalized cone resistance was less than 50 the minimum (liquefied) shear strength ratio was less than 0.1, which is consistent with the work of Yoshimine et al. (1999). No case history had a back-calculated liquefied shear strength greater than 0.12. The relationship suggested by Yoshimine et al (1998) also matches the case history data reviewed by Olsen and Stark, although the extrapolation beyond  $q_{c1N} = 65$  is different.

The relationship suggested by Yoshimine et al. (1999) tends to produce conservative lower bound values compared to the Olsen and Stark (2002) method for a normalized cone resistance  $q_{c1N} < 40$ , but both give the same result at  $q_{c1N} = 50$ . Both methods are appropriate for young uncemented, essentially normally consolidated soils where the vertical effective stress is less than about 300 kPa and  $I_c \leq 2.6$ .

Sandy soils that have a minimum undrained shear strength ratio in simple shear of around 0.30 or higher are generally strain hardening and not brittle. Sandy soils that have a minimum (liquefied) undrained shear strength ratio of around 0.10 or less are strain softening and can have high brittleness. Hence, a value of  $s_{u(min)}/\sigma'_{vo} = 0.10$  represents an approximate simplified boundary between soils that can show significant strain softening in undrained simple shear and soils that show little or no strain softening. For simple shear direction of loading, this boundary can be represented by a normalized clean sand equivalent CPT value of  $(q_{c1N})_{cs} = 50$ . Hence, a value of  $(q_{c1N})_{cs} < 50$  can be used as an approximate simplified criteria to estimate if a soil may be brittle and strain softening in undrained simple shear. If the soil is cohesionless (i.e. non-plastic,  $I_c < 2.6$ ) the loss of strength could occur rapidly over small strains (i.e. brittle response). If the soil is cohesive the loss of strength could occur more slowly, depending on the degree of sensitivity and plasticity of the soil. Highly sensitive clays (quick clays) can lose their strength rapidly resulting in flow slides. Less sensitive clays tend to deform gradually without a flow slide. The suggested simplified criteria of  $(q_{c1N})_{cs} < 50$  is based on simple shear direction of loading and a soil response that implies significant strain softening. Given that the relationship suggested by Yoshimine et al (1999) conservatively captures most of the available data, a simplified criteria of  $(q_{c1N})_{cs} < 50$  is considered appropriate for screening purposes:

The criteria of  $(q_{c1N})_{cs} = 50$  has been included on the attached CPT liquefaction plots as a simple means to evaluate the potential for flow liquefaction. However, for flow liquefaction to occur it requires a trigger event to initiate strain softening and a sufficient volume of strain softening soil to form a kinematically admissible mechanism. The extent of weak potentially strain softening soil layers can be evaluated using several CPT profiles.

For the example shown there are zones where  $(q_{c1N})_{cs} < 50$ , indicating there is sandy soil which is loose and brittle, which would strain soften in simple shear loading.

### Conclusions

The example CPT profiles illustrate how the CPT can be used to evaluate liquefaction potential in a cost effective, continuous manner. Experience has shown that the CPT provides reliable, repeatable, cost effective, and, continuous profiles of penetration resistance that can be analyzed to provide estimates of liquefaction potential as well as estimates of post-earthquake settlements and lateral spreads. The CPT data can also be used to estimate the potential for flow liquefaction and instability.

Yours truly,

P.K. Robertson, Ph.D., P. Eng.

# Evaluating Soil Liquefaction using the CPT

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University of Alberta  
Canada

## Introduction

Soil liquefaction is a major concern for structures constructed with or on sand or sandy soils. The major earthquakes of Niigata in 1964 and Kobe in 1995 have illustrated the significance and extent of damage caused by soil liquefaction. Soil liquefaction is also a major design problem for large sand structures such as mine tailings impoundment and earth dams.

To evaluate the potential for soil liquefaction it is important to determine the soil stratigraphy and in-situ state of the deposits. The CPT is an ideal in-situ test to evaluate the potential for soil liquefaction because of its repeatability, reliability, continuous data and cost effectiveness. This paper presents a summary of the application of the CPT to evaluate soil liquefaction. Further details are contained in a series of papers (Robertson and Wride, 1998; Youd et al., 2001; Zhang et al., 2002; Zhang et al., 2004).

## Definition of Soil Liquefaction

Several phenomena are described as soil liquefaction, hence, a series of definitions are provided to aid in the understanding of the phenomena.

### *Cyclic (softening) Liquefaction*

- Requires undrained cyclic loading during which shear stress reversal occurs or zero shear stress can develop.
- Requires sufficient undrained cyclic loading to allow effective stresses to reach essentially zero.
- Deformations during cyclic loading can accumulate to large values, but generally stabilize shortly after cyclic loading stops. The resulting movements are due to external causes and occur mainly during the cyclic loading.
- Can occur in almost all saturated sands provided that the cyclic loading is sufficiently large in magnitude and duration.
- Clayey soils can experience cyclic liquefaction but deformations are generally small due to the cohesive strength at zero effective stress. Rate effects (creep) often control deformations in cohesive soils.

### *Flow Liquefaction*

- Applies to strain softening soils only.
- Requires a strain softening response in undrained loading resulting in approximately constant shear stress and effective stress.
- Requires in-situ shear stresses to be greater than the residual or minimum undrained shear strength
- Either monotonic or cyclic loading can trigger flow liquefaction.
- For failure of a soil structure to occur, such as a slope, a sufficient volume of material must strain soften. The resulting failure can be a slide or a flow depending on the material characteristics and ground geometry. The resulting movements are due to internal causes and can occur after the trigger mechanism occurs.
- Can occur in any metastable saturated soil, such as very loose fine cohesionless deposits, very sensitive clays, and loess (silt) deposits.

Note that strain softening soils can also experience cyclic liquefaction depending on ground geometry. Figure 1 presents a flow chart (Robertson and Wride, 1998) to clarify the phenomena of soil liquefaction.

If a soil is strain softening, flow liquefaction is possible if the soil can be triggered to collapse and if the gravitational shear stresses are larger than the minimum undrained shear strength. The trigger can be either monotonic or cyclic. Whether a slope or soil structure will fail and slide will depend on the amount of strain softening soil relative to strain hardening soil within the structure, the brittleness of the strain softening soil and the geometry of the ground. The resulting deformations of a soil structure with both strain softening and strain hardening soils will depend on many factors, such as distribution of soils, ground geometry, amount and type of trigger mechanism, brittleness of the strain softening soil and drainage conditions. Examples of flow liquefaction failures are the Aberfan flow slide (Bishop, 1973), Zealand submarine flow slides (Koppejan et al., 1948) and the Stava tailings dam failure. In general, flow liquefaction failures are not common, however, when they occur, they typically take place rapidly with little warning and are usually catastrophic. Hence, the design against flow liquefaction should be carried out cautiously.

If a soil is strain hardening, flow liquefaction will not occur. However, cyclic liquefaction can occur due to cyclic undrained loading. The amount and extent of deformations during cyclic loading will depend on the density of the soils, the magnitude and duration of the cyclic loading and the extent to which shear stress reversal occurs. If extensive shear stress reversal occurs and the magnitude and duration of the cyclic loading are sufficiently large, it is possible for the effective stresses to essentially reach zero with resulting possible large deformations. Examples of cyclic liquefaction were common in the major earthquakes in Niigata in 1964 and Kobe in 1995 and manifest in the form of sand boils, damaged lifelines (pipelines, etc.) lateral spreads, slumping of embankments, ground settlements, and ground surface cracks. If cyclic liquefaction occurs and drainage paths are restricted due to overlying less permeable layers, the sand immediately beneath the less permeable soil can loosen

due to pore water redistribution, resulting in possible subsequent flow liquefaction, given the right geometry.

The three main concerns related to soil liquefaction are generally:

- Will an event (e.g. a design earthquake) trigger significant zones of liquefaction?
- What displacements will result, if liquefaction is triggered?
- What will be the resulting residual (minimum) undrained shear strength, if a soil is potentially strain softening (and is triggered to liquefy)?

This paper describes how the CPT can be used to evaluate the potential for an earthquake to trigger cyclic liquefaction, and, then how to estimate post-earthquake displacements (vertical and lateral). Finally, a method is described on how to estimate the minimum undrained shear strength, using the CPT, which could result if the soil is potentially strain softening (flow liquefaction).

### Cyclic Liquefaction

Most of the existing work on cyclic liquefaction has been primarily for earthquakes. The late Prof. H.B. Seed and his co-workers developed a comprehensive methodology to estimate the potential for cyclic liquefaction due to earthquake loading. The methodology requires an estimate of the cyclic stress ratio (CSR) profile caused by the design earthquake and the cyclic resistance ratio (CRR) of the ground. If the CSR is greater than the CRR cyclic liquefaction can occur. The CSR is usually estimated based on a probability of occurrence for a given earthquake. A site-specific seismicity analysis can be carried out to determine the design CSR profile with depth. A simplified method to estimate CSR was also developed by Seed and Idriss (1971) based on the maximum ground surface acceleration ( $a_{max}$ ) at the site. The simplified approach can be summarized as follows:

$$[1] \quad CSR = \frac{\tau_{av}}{\sigma'_{vo}} = 0.65 \left[ \frac{a_{max}}{g} \right] \left( \frac{\sigma_{vo}}{\sigma'_{vo}} \right) r_d$$

Where  $\tau_{av}$  is the average cyclic shear stress;  $a_{max}$  is the maximum horizontal acceleration at the ground surface;  $g = 9.81\text{m/s}^2$  is the acceleration due to gravity;  $\sigma_{vo}$  and  $\sigma'_{vo}$  are the total and effective vertical overburden stresses, respectively; and  $r_d$  is a stress reduction factor which is dependent on depth. The factor  $r_d$  can be estimated using the following bi-linear function, which provides a good fit to the average of the suggested range in  $r_d$  originally proposed by Seed and Idriss (1971):

$$[2] \quad r_d = 1.0 - 0.00765z \\ \text{if } z < 9.15 \text{ m} \\ = 1.174 - 0.0267z$$

if  $z = 9.15$  to  $23$  m

Where  $z$  is the depth in metres. These formulae are approximate at best and represent only average values since  $r_d$  shows considerable variation with depth.

Seed et al., (1985) also developed a method to estimate the cyclic resistance ratio (CRR) for clean sand with level ground conditions based on the Standard Penetration Test (SPT). Recently the CPT has become more popular to estimate CRR, due to the continuous, reliable and repeatable nature of the data (Youd et al., 2001).

In recent years, there has been an increase in available field performance data, especially for the CPT (Robertson and Wride, 1998). The recent field performance data have shown that the existing CPT-based correlation by Robertson and Campanella (1985) for clean sands is generally good. Based on discussions at the 1996 NCEER workshop (NCEER, 1997), the curve by Robertson and Campanella (1985) has been adjusted slightly at the lower end. The resulting recommended CPT correlation for clean sand is shown in Figure 2 and can be estimated using the following simplified equations:

$$[3] \quad CRR_{7.5} = 93 \left[ \frac{(q_{c1N})_{cs}}{1000} \right]^3 + 0.08$$

if  $50 \leq (q_{c1N})_{cs} \leq 160$

$$CRR_{7.5} = 0.833 \left[ \frac{(q_{c1N})_{cs}}{1000} \right]^3 + 0.05$$

if  $(q_{c1N})_{cs} < 50$

The field observations used to compile the curve in Figure 2 are based primarily on the following conditions:

- Holocene age, clean sand deposits
- Level or gently sloping ground
- Magnitude  $M = 7.5$  earthquakes
- Depth range from 1 to 15 m (85% is for depths  $< 10$  m)
- Representative average CPT values for the layer considered to have experienced cyclic liquefaction.

Caution should be exercised when extrapolating the CPT correlation to conditions outside the above range. An important feature to recognize is that the correlation is based primarily on average values for the inferred liquefied layers. However, the correlation is often applied to all measured CPT values, which include low values below the average. Therefore, the correlation can be conservative in variable deposits where a small part of the CPT data can indicate possible liquefaction.

It has been recognized for some time that the correlation to estimate  $CRR_{7.5}$  for silty sands is different than that for clean sands. Typically a correction is made to determine an equivalent clean sand penetration resistance based on grain characteristics, such as fines content, although the corrections are due to more than just fines content.

One reason for the continued use of the SPT has been the need to obtain a soil sample to determine the fines content of the soil. However, this has been offset by the generally poor repeatability and reliability of the SPT data. It is now possible to estimate grain characteristics directly from the CPT. Robertson and Wride (1998) suggest estimating an equivalent clean sand cone penetration resistance,  $(q_{c1N})_{cs}$  using the following:

$$[4] \quad (q_{c1N})_{cs} = K_c (q_{c1N})$$

where  $K_c$  is a correction factor that is a function of grain characteristics of the soil.

Robertson and Wride (1998) suggest estimating the grain characteristics using the soil behavior chart by Robertson (1990) (see Figure 3) and the soil behavior type index,  $I_c$ , where;

$$[5] \quad I_c = \left[ (3.47 - \log Q)^2 + (\log F + 1.22)^2 \right]^{0.5}$$

$$\text{where } Q = q_{c1N} = \left( \frac{q_c - \sigma_{vo}}{P_{a2}} \right) \left( \frac{P_a}{\sigma'_{vo}} \right)^n$$

is the normalized CPT penetration resistance, dimensionless;  $n$  = stress exponent;  $F = f_s / [(q_c - \sigma_{vo})] \times 100\%$  is the normalized friction ratio, in percent;  $f_s$  is the CPT sleeve friction stress;  $\sigma_{vo}$  and  $\sigma'_{vo}$  are the total effective overburden stresses, respectively;  $P_a$  is a reference pressure in the same units as  $\sigma'_{vo}$  (i.e.  $P_a = 100$  kPa if  $\sigma'_{vo}$  is in kPa); and  $P_{a2}$  is a reference pressure in the same units as  $q_c$  and  $\sigma_{vo}$  (i.e.  $P_{a2} = 0.1$  MPa if  $q_c$  and  $\sigma_{vo}$  are in MPa). Robertson and Wride (1998) used a form of  $q_{c1N}$  that did not subtract the total vertical stress ( $\sigma_{vo}$ ) from  $q_c$  and used  $n = 0.5$ . The more correct approach is the full form shown in equation 5.

The soil behaviour type chart by Robertson (1990) shown in Figure 3, uses a normalized cone penetration resistance ( $Q$ ) based on a simple linear stress exponent of  $n = 1.0$ , whereas the chart recommended here for estimating CRR in sand is based on a normalized cone penetration resistance ( $q_{c1N}$ ) based on a stress exponent  $n = 0.5$ . Olsen and Malone (1988) correctly suggested a normalization where the stress exponent ( $n$ ) varies from around 0.5 in sands to 1.0 in clays. The procedure using  $n = 1.0$  was recommended by Robertson and Wride (1998) for soil classification in clay type soils when  $I_c > 2.6$ . However, in sandy soils when  $I_c \leq 2.6$ , Robertson and Wride (1998) recommended that data being plotted on the chart be modified by using  $n = 0.5$ .

The simplified normalization suggested by Robertson and Wride (1998) is easy to apply but produces a somewhat discontinuous variation of the stress exponent,  $n$ . To produce



a smoother variation of the stress exponent the following modified method is recommended.

Assume an initial stress exponent  $n = 1.0$  and calculate  $Q$  and  $F$  and then  $I_c$ . Then:

$$[6] \quad \begin{array}{ll} \text{If } I_c < 1.64 & n = 0.5 \\ \text{If } I_c > 3.30 & n = 1.0 \\ \text{If } 1.64 < I_c < 3.30 & n = (I_c - 1.64) 0.3 + 0.5 \end{array}$$

Iterate until the change in the stress exponent,  $\Delta n < 0.01$ .

When the in-situ vertical effective stress ( $\sigma'_{vo}$ ) exceeds 300 kPa assume  $n = 1.0$  for all soils. This avoids the need to make further corrections for high overburden stresses (i.e.  $K_\sigma$ ).

The recommended relationship between  $I_c$  and the correction factor  $K_c$  is given by the following:

$$[7] \quad K_c = 1.0 \quad \text{if } I_c \leq 1.64$$

$$K_c = -0.403 I_c^4 + 5.581 I_c^3 - 21.63 I_c^2 + 33.75 I_c - 17.88 \quad \text{if } I_c > 1.64$$

The correction factor,  $K_c$ , is approximate since the CPT responds to many factors, such as, soil plasticity, fines content, mineralogy, soil sensitivity, age and stress history. However, in general, these same factors influence the  $CRR_{7.5}$  in a similar manner. Caution should be used when applying the relationship to sands that plot in the region defined by  $1.64 < I_c < 2.36$  and  $F < 0.5\%$  so as not to confuse very loose clean sands with sands containing fines. In this zone, it is suggested to set  $K_c = 1.0$ .

Soils with  $I_c > 2.6$  fall into the clayey silt, silty clay and clay regions of the CPT soil behavior chart and, in general, are essentially non-liquefiable. Samples should be obtained and liquefaction evaluated using other criteria based on index parameters (Youd et al., 2001). Soils that fall in the lower left region of the CPT soil behavior chart defined by  $I_c > 2.6$  but  $F < 1.0\%$  can be sensitive fine-grained soils and hence, possibly susceptible to both cyclic and flow liquefaction. Fine-grained, cohesive (clay) soils can develop significant strains and deformations during earthquake loading if the level and duration of shaking are sufficient to overcome the peak undrained resistance of the soil. If that were to happen and sufficient movement were to accrue, the strength of the soil could reduce to its residual or remolded strength depending on the amount of strain required to soften the soil.

The full methodology to estimate  $CRR_{7.5}$  from the CPT is summarized in Figure 4.

Recent publications have attempted to update the relationship between normalized cone resistance ( $q_{cIN}$ ) and  $CRR$  (Figure 2). However, these efforts have limited additional value since the relationship is based on average values in layers that were thought to have liquefied. This requires considerable judgment and is subject to much uncertainty. It is now time to move away from the boundary curve diagrams represented in Figure 2.

The preferred approach is to continue to refine the total integrated CPT-based approach based on case histories using the full CPT and CRR profiles. Ideally case histories should include extensive CPT data, samples and observations of ground deformations (both surface and subsurface) so that a full evaluation can be made of the impact of the earthquake loading and where the deformations occurred. This can be important, since deformations can occur where full liquefaction may not have taken place (i.e. factor of safety greater than 1). Currently most publications take case history data and estimate the layer that experienced liquefaction and then assign an average penetration resistance value to that layer. In this way each case history can be represented by one data point. However, each case history should be represented by the many hundred data points contained in the full CPT profile. Often the ground profiles are complex and liquefaction may not have occurred in one well-defined single layer. Liquefaction often occurs in multiple layers, which can only be observed in the full CPT profile. It can be overly simplistic to represent a complex ground profile and case history by one data point on a boundary curve like Figure 2. These curves have served as an excellent starting point in the development of the current simplified CPT and SPT liquefaction methods now available, however, it is now time to leave these highly simplistic curves and progress using data captured in the full soil profile.

The factor of safety against liquefaction is defined as:

$$[8] \quad \text{Factor of Safety, FS} = \frac{\text{CRR}_{7.5}}{\text{CSR}} \text{MSF}$$

where MSF is the Magnitude Scaling Factor to convert the  $\text{CRR}_{7.5}$  for  $M = 7.5$  to the equivalent CRR for the design earthquake. The recommended MSF is given by:

$$[9] \quad \text{MSF} = \frac{174}{M^{2.56}}$$

The above recommendations are based on the NCEER Workshop in 1996 (Youd et al., 2000).

An example of the CPT method to evaluate cyclic liquefaction is shown on Figure 5 for the Moss Landing site that suffered cyclic liquefaction during the 1989 Loma Prieta earthquake in California (Boulanger et al., 1995).

A major advantage of the CPT approach is the continuous and reliable nature of the data. CPT data are typically collected every 5 cm (2 inches). This means that data points are collected at the interface between layers, such as between clay and sand. During this transition, the CPT data points do not accurately capture the correct soil response since the penetration resistance is moving from either low to high values or vis-a-versa. In these thin transition zones the CPT-based liquefaction method can predict low values of CRR. This is illustrated in Figure 5 at depths of around 6m and 10m, where there are clear interface boundaries between sand and clay. At these

locations there a few data points that indicate low values of CRR and hence liquefaction. These thin interface zones are easy to identify and account for in the interpretation. In the following sections, methods to estimate post-earthquake ground deformations will be presented and discussed. Using these CPT-based methods, it is simple to identify the thin interface transition zones and remove.

A key advantage of the CPT based liquefaction method is that continuous profiles can be calculated quickly, which allows the engineer time to study the profile in detail and apply engineering judgment where appropriate. The CPT-based liquefaction method is a simplified approach and is hence conservative. The method was developed from the limit boundary curve in Figure 2 that was developed using average values but the resulting method is applied using all data points. Also, the continuous CPT data predicts low values of CRR in the thin interface transitions zones, as described above.

Juang et al. (1999) has shown that the Robertson and Wride CPT-based liquefaction method has the same level of conservatism as the Seed et al SPT-based liquefaction method. This conclusion was supported by the NCEER workshop (Youd et al., 2001).

### **Post-earthquake Deformations**

The CPT-based method described above, can provide continuous profiles of CRR and Factor of Safety for given design earthquake loading. However, Factor of Safety is not always the most meaningful means to evaluate liquefaction potential. For most projects a more meaningful evaluation of the effect of a design earthquake on a given project is to estimate the ground deformations that may result from the earthquake. Ground deformations that follow earthquake loading are either vertical settlements or lateral deformations. Although the Factor of Safety due to a design earthquake may be less than 1.0, the resulting deformations may be either acceptable for the project or can be accommodated with appropriate design of the structures.

#### ***Liquefaction induced Vertical ground settlements***

Liquefaction-induced ground settlements are essentially vertical deformations of surficial soil layers caused by the densification and compaction of loose granular soils following earthquake loading. Several methods have been proposed to calculate liquefaction-induced ground deformations, including numerical and analytical methods, laboratory modeling and testing, and field-testing-based methods. The expense and difficulty associated with obtaining and testing high quality samples of loose sandy soils may only be feasible for high-risk projects where the consequences of liquefaction may result in severe damage and large costs. Semi-empirical approaches using data from field tests are likely best suited to provide simple, reliable and direct methods to estimate liquefaction-induced ground deformations for low to medium risk projects, and also to provide preliminary estimates for higher risk projects. Zhang et al. (2002) proposed a simple semi-empirical method using the CPT to estimate liquefaction induced ground settlements for level ground.

For sites with level ground, far from any free face (e.g., river banks, seawalls), it is reasonable to assume that little or no lateral displacement occurs after the earthquake, such that the volumetric strain will be equal or close to the vertical strain. If the vertical strain in each soil layer is integrated with depth using Equation [10], the result should be an appropriate index of potential liquefaction-induced ground settlement at the CPT location due to the design earthquake,

$$[10] \quad S = \sum_{i=1}^j \varepsilon_{vi} \Delta z_i$$

Where:  $S$  is the calculated liquefaction-induced ground settlement at the CPT location;  $\varepsilon_{vi}$  is the post-liquefaction volumetric strain for the soil sub-layer  $i$ ;  $\Delta z_i$  is the thickness of the sub-layer  $i$ ; and  $j$  is the number of soil sub-layers.

The method suggested by Zhang et al (2002) is based laboratory results (Ishihara and Yoshimine, 1992) to estimate liquefaction induced volumetric strains for sandy and silty soils, as shown on Figure 6. The procedure can be illustrated using a CPT profile from the Marina District site in California. Figure 7 illustrates the major steps in the CPT-based liquefaction potential analysis and shows the profiles of measured CPT tip resistance  $q_c$ , sleeve friction  $f_s$ , soil behavior type index  $I_c$ , cyclic resistance ratio CRR & cyclic stress ratio CSR, and factor of safety against liquefaction FS, respectively. The data in Figures 7a and 7b can be directly obtained from the CPT sounding. Figures 7c, 7d and 7e show the results calculated based on the Robertson and Wride procedure shown in Figure 4. Note that, according to Robertson and Wride's approach, CRR is not calculated when the soil behavior type index is greater than 2.6. These soils are assumed to be non-liquefiable in Robertson and Wride's approach.

The four key plots for estimating liquefaction induced ground settlements by the CPT-based method are presented in Figure 8. Figures 8a to 8d show the profiles of equivalent clean sand normalized tip resistance  $(q_{c1N})_{cs}$ , factor of safety FS, post-liquefaction volumetric strain  $\varepsilon_v$ , and liquefaction induced ground settlement  $S$ , respectively. Data in Figures 8a and 8b are from the liquefaction potential analysis. Data in Figure 8c are calculated from the results presented by Ishihara and Yoshimine, (1992) and shown in Figure 6. The settlement shown in Figure 8d is obtained using Equation [10] and the volumetric strains from Figure 8c.

The method by Zhang et al (2002) was evaluated using liquefaction-induced ground settlements from the Marina District and Treasure Island case histories damaged by liquefaction during the 1989 Loma Prieta earthquake. Good agreement between the calculated and measured liquefaction-induced ground settlements was found. Although further evaluations of the method are required with future case history data from different earthquakes and ground conditions, as they become available, it is suggested that the CPT-based method may be used to estimate liquefaction-induced settlements for low to medium risk projects and also provide preliminary estimates for higher risk projects.

### *Effects of other major factors on calculated settlements*

#### **Maximum surface acceleration**

The amplification of earthquake motions is a complex process and is dependent on soil properties, thickness, frequency content of motions and local geological settings. For a given earthquake and geological setting, the amplification increases with the increase of soil compressibility and with soil thickness. Maximum surface acceleration at a site is one important parameter used in evaluating liquefaction potential of sandy soils. However, its determination is difficult without recorded accelerographs for a given earthquake because it likely varies with soil stratigraphy, soil properties, earthquake properties, the relative location of the site to the epicenter and even ground geometry. Ground response analysis may help to solve the problem but still leave some uncertainty in the results. Zhang et al. (2002) illustrated the importance of maximum surface acceleration and showed that calculated settlements are not linearly proportional to maximum surface acceleration beyond certain values since the calculated volumetric strains reach limiting values (Fig. 6). Using the Zhang et al (2002) approach it is possible to calculate settlements as a function of surface acceleration as a means to evaluate the sensitivity of the approach to the design earthquake.

#### **Fines content or mean grain size**

Although there are many practical situations where liquefaction settlements need to be estimated for sands with little silt to silty-sands, the volumetric strains shown in Figure 6 are applicable to saturated clean sands only. It is therefore necessary to consider the effects of fines content on post-liquefaction volumetric strains and liquefaction potential.

Volumetric strains tend to increase with increasing mean grain size at a given relative density (Lee and Albaisa 1974). Since increasing the fines content of a sand will result in a decrease of the mean grain size, it is postulated that post-liquefaction volumetric strains would decrease with increasing fines content in sands at a given relative density.

Silty sands have been found to be considerably less vulnerable to liquefaction than clean sands with similar SPT blow-counts (Iwasaki et al. 1978; Tatsuoka et al. 1980; Tokimatsu and Yoshimi 1981; Zhou 1981; et al.). Consequently, modification factors to SPT blow-counts or cyclic resistance ratio for sands with different fines contents or mean grain sizes had been widely used in liquefaction potential analyses (Seed and Idriss 1982; Robertson and Campanella 1985; Seed et al. 1985; Robertson and Wride 1998).

Zhang et al (2002) used the equivalent clean sand normalized CPT tip resistance  $(q_{c1N})_{cs}$  to account for the effects of sand grain characteristics and apparent fines content on CRR.  $(q_{c1N})_{cs}$  is also used to estimate post-liquefaction volumetric strains for sands with fines. This approach assumes that both the liquefaction resistance and post-liquefaction deformations of silty sandy soils can be quantified using the same method and formulas as for clean sands provided that the equivalent clean sand normalized cone penetration resistance,  $(q_{c1N})_{cs}$ , is used. This implies that no further modification is required for the effects of fines content or mean grain size if  $(q_{c1N})_{cs}$  is used to estimate the liquefaction induced settlements of sandy soils including silty sands. Because  $(q_{c1N})_{cs}$  will increase

with increase of fines content with sands for a given cone tip resistance, the calculated post-liquefaction volumetric strains will decrease with increase of fines content for a given factor of safety. This approach appears to indirectly account partially or wholly for the effect of grain characteristics on post-liquefaction volumetric strains and provides the same trend as observed by Lee and Albaisa (1974).

#### **Transitional zone or thin sandy soil layers**

It is recognized that transitional zones between soft clay layers and stiff sandy soil layers influence the calculated liquefaction-induced settlements. However, the influence of the transitional zones on calculated  $(q_{c1N})_{cs}$ , and FS has been partially counteracted implicitly in the Robertson and Wride method. Generally, the measured tip resistance in a sandy soil layer close to a soft soil layer (usually a clayey soil layer) is smaller than the "actual" tip resistance (if no layer interface existed) and the resultant friction ratio is greater than the "actual" friction ratio due to the influence of the soft soil layer. As a result, the calculated value of  $I_c$  will increase, and therefore the correction factor  $K_c$ ,  $(q_{c1N})_{cs}$ , and FS will increase as well.  $(q_{c1N})_{cs}$  and FS may be close to the "true" values in a same sandy soil layer that is not influenced by the soft soil layer. Therefore, the calculated ground settlements would be close the "actual" values because of this implicit correction incorporated with the Robertson and Wride method.

Zhang et al. (2001) took no correction in an attempt to quantify the influences of both the transitional zones and thin sandy layers on the tip resistance, yet achieved good agreement with the limited case history results. Making no correction for transitional zones and thin layers is conservative when estimating liquefaction potential and liquefaction related deformations. Further investigation is required to quantify the influence of transitional zones or thin sandy soil layers on calculated FS and liquefaction-induced ground settlements.

#### **Three dimensional distribution of liquefied soil layers**

The thickness, depth and lateral distribution of liquefied layers may play an important role on ground surface settlements. Liquefaction of a relatively thick but deep sandy soil (see Figure 9a) may have minimal effect on the performance of an overlying structure founded on shallow foundations. However, liquefaction of a near surface thin layer of soil (Figure 9b) may have major implications on the performance of the same structure.

Ishihara (1985) provided some guidance on the effect of thickness and depth to the liquefied layer on potential settlements that may be reasonable provided that the site is not susceptible to ground oscillation or lateral spread (Youd and Garriss 1995, O'Rourke and Pease 1997). Gilstrap (1998) concluded that Ishihara's relationship for predicting surface effects may be oversimplified. As well, the application of Ishihara's criteria in practice for cases with multiple liquefied layers (Figure 9c) is not clear.

The lateral extent of liquefied layers may also have an effect on ground surface settlements. A small locally liquefied soil zone with limited lateral extent (Figure 9d) would have limited extent of surface manifestation than that for a horizontally extensive liquefied soil zone with the same soil properties and vertical distribution of the liquefied layer. On the other hand, the locally liquefied soil zone may be more damaging to the

engineered structures and facilities due to the potential large differential settlements. However, no quantitative study has been reported for the effect of lateral extent of liquefied layers on ground surface settlements.

Neglecting the effect of three-dimensional distribution of liquefied layers on ground surface settlements may result in over-estimating liquefaction-induced ground settlements for some sites. Engineering judgement is needed to avoid an overly conservative design. Case histories from previous earthquakes have indicated that little or no surface manifestation was observed for cases where the depth from ground surface to the top of the liquefied layer was greater than 20 m. Care is required to detect local zones of soil that may liquefy and to estimate the potential differential settlements that may occur.

#### Factor $K_c$

Robertson and Wride (1998) recommended the factor  $K_c$  be set equal to 1.0 rather than using  $K_c$  of 1.0 to 2.14 when the CPT data plot in the zone defined by  $1.64 < I_c < 2.36$  and  $F < 0.5\%$  to avoid confusion of very loose clean sands with denser sands containing fines. However, if the CPT data of a dense sand with fines plots in the zone ( $1.64 < I_c < 2.36$  and  $F < 0.5\%$ ), the calculated  $(q_{c1N})_{cs}$  value for the dense sand would be reduced by one-half. Although this recommendation is conservative for evaluating liquefaction potential of sandy soils, it may result in over-estimating of liquefaction-induced ground settlements for sites with denser sands containing fines that fit in that zone. This seems to be true for some of the CPT soundings in the two case histories studied by Zhang et al (2002). For example, based on soil profiles, CPT profiles, and engineering judgement, a portion of the soil that should have been assessed as dense sand containing fines, was classified as very loose clean sand with  $K_c$  equal to 1.0.

Zhang et al (2002) showed that when the settlements for the two case histories were recalculated without following the recommendation of  $K_c$  equal to 1.0 for  $1.64 < I_c < 2.36$  and  $F < 0.5\%$ , there was almost no effect for the western and central parts of Marina District and only small (up to 14%) effects for Treasure Island. However the calculated settlements for the eastern part of Marina District were reduced by a factor of 2.6 without the recommendation for  $K_c$ . The effect of this recommendation on calculated ground settlements depends on the amount of the soils that fit in the zone defined by  $1.64 < I_c < 2.36$  and  $F < 0.5\%$  within a soil profile for a site studied. If a large amount of the soils fit in this zone, the effect would be much more significant than that for the two case history sites studied above. Soil sampling is therefore recommended to clarify soil properties for sites where a large amount of soil plots in the zone  $1.64 < I_c < 2.36$  and  $F < 0.5\%$ .

#### Cutoff of $I_c$ equal to 2.6

A cutoff of  $I_c$  equal to 2.6 is used to distinguish sandy and silty soils from clayey soils, which are believed to be non-liquefiable (Robertson and Wride, 1998). Gilstrap (1998) concluded that the  $I_c$  cutoff of 2.6 recommended by Robertson and Wride (1998) is generally reliable for identifying clayey soils, but noticed that 20% to 50% of the samples with  $I_c$  between 2.4 to 2.6 were classified as clayey soils based on index tests. This implies that the cutoff of  $I_c$  equal to 2.6 appears slightly conservative.

Zhang et al (2002) investigated the sensitivity of the calculated settlements to this cutoff for the two case histories using a cutoff of  $I_c$  equal to 2.5. The calculated settlements with the cutoff of  $I_c$  equal to 2.5 were slightly smaller than with the cutoff of 2.6. For the two cases, only a small portion of the soil in the profiles had  $I_c$  ranging from 2.5 to 2.6, thus the use of a cutoff of  $I_c$  equal to 2.6 does not greatly overestimate the settlements.

Neglecting the influence of the recommendation for  $K_c$  and the cutoff line of  $I_c$  equal to 2.6 on the calculated ground settlements is conservative. However, soil sampling is recommended to avoid unnecessary overestimation of liquefaction-induced ground settlements for some sites where a large amount of the soils have a calculated  $I_c$  close to 2.6 or/and fit in the zone defined by  $1.64 < I_c < 2.36$  and  $F < 0.5\%$ .

### *Liquefaction induced Lateral Displacements*

Generally, liquefaction-induced ground failures include flow slides, lateral spreads, ground settlements, ground oscillation, and sand boils. Lateral spreads are the pervasive types of liquefaction-induced ground failures for gentle slopes or for nearly level (or gently inclined) ground with a free face (e.g., river banks, road cuts).

Several methods have been proposed to estimate liquefaction-induced lateral ground displacements including numerical models, laboratory tests, and field-test-based methods. Challenges associated with sampling loose sandy soils limit the applications of numerical and laboratory testing approaches in routine practice. Field-test-based methods are likely best suited to provide simple direct methods to estimate liquefaction-induced ground deformations for low- to medium-risk projects and to provide preliminary estimates for high-risk projects.

One-g shake table tests (e.g., Sasaki et al. 1991, Yasuda et al. 1992) and centrifuge model tests (e.g., Abdoun 1997, Taboada and Dobry 1998) have been conducted to investigate the mechanisms of liquefaction-induced ground lateral spreads. These tests support the hypothesis that lateral spreads result from distributed residual shear strains throughout the liquefied layers. The residual shear strains in liquefied layers are primarily a function of: (a) maximum cyclic shear strains  $\gamma_{max}$ , and (b) biased insitu static shear stresses. In this paper,  $\gamma_{max}$  refers to the maximum amplitude of cyclic shear strains that are induced during undrained cyclic loading for a saturated sandy soil without biased static shear stresses in the direction of cyclic loading. Biased insitu static shear stresses are mainly controlled by ground geometry at the site (e.g., ground slope, free face height, and the distance to a free face). The thickness of liquefied layers will also influence the magnitude of lateral displacements, with greater lateral displacements for thicker liquefied layers. Both  $\gamma_{max}$  and the thickness of liquefied layers are affected by soil properties and earthquake characteristics.

Zhang et al (2004) suggested a semi-empirical method based on CPT results to estimate the lateral displacement using the combined results from laboratory tests with case history data from previous earthquakes. The method captures the mechanisms of liquefaction-induced lateral spreads and characterizes the major factors controlling



lateral displacements. Application of the method is simple and can be applied with only a few calculations following the CPT-based liquefaction potential analysis. The approach may be suitable to estimate the magnitude of lateral displacements associated with liquefaction-induced lateral spread for gently sloping (or level) ground with or without a free face for low to medium-risk projects, or to provide preliminary estimates for higher risk projects.

The Zhang et al (2004) method uses the laboratory results presented by Ishihara and Yoshimine (1992) to estimate the maximum shear strains,  $\gamma_{max}$ , as shown on Figure 10. Zhang et al (2004) used the correlation between  $D_r$  and normalized cone tip resistance ( $q_{cIN}$ ) suggested by Tatsuoka et al. (1990):

$$[11] \quad D_r = -85 + 76 \log (q_{cIN}) \quad (q_{cIN} \leq 200)$$

Where:  $q_{cIN}$  is the normalized CPT tip resistance corrected for effective overburden stresses corresponding to 100 kPa (Robertson and Wride, 1998).

This correlation provides slightly smaller and more conservative estimates of relative density than the correlation by Jamiolkowski et al. (1985) when  $q_{cIN}$  is less than about 100.

Integrating the calculated  $\gamma_{max}$  values with depth will produce a value that is defined as the lateral displacement index, (LDI):

$$[12] \quad LDI = \int_0^{Z_{max}} \gamma_{max} dz$$

where  $Z_{max}$  is the maximum depth below all the potential liquefiable layers with a calculated FS < 2.0.

Using case history results and ground geometric parameters to characterize the ground geometry, the lateral displacement (LD) can be estimated.

**For gently sloping ground without a free-face;** Zhang et al (2004) suggested the following relationship based on the available case histories;

$$[13] \quad \frac{LD}{LDI} = S + 0.2 \quad (\text{for } 0.2\% < S < 3.5\%)$$

Where: S is the ground slope as a percentage.

**For level ground conditions with a free-face;** Zhang et al (2004) suggested the following relationship based on the available case histories:

$$[14] \quad \frac{LD}{LDI} = 6 \cdot \left( \frac{L}{H} \right)^{-0.8} \quad (\text{for } 4 < L/H < 40)$$

Where: L is the horizontal distance from the toe of a free-face,  
H is the height of the free-face.

Level ground is taken as ground with a slope less than 0.15%.

The Zhang et al (2004) approach is recommended for use within the ranges of earthquake properties, moment magnitude ( $M_w$ ) between 6.4 and 9.2, and, peak surface acceleration ( $a_{max}$ ) between 0.19g and 0.60 g, and free face heights less than 18 m. The case history data used for developing the approach, especially for gently sloping ground without a free face, were dominantly from two Japanese case histories associated with the 1964 Niigata and 1983 Nihonkai-Chubu earthquakes, where the liquefied soils were mainly clean sand only. The values for the geometric parameters used in developing the approach were within limited ranges, as specified in Equations [13] and [14]. It is recommended that the approach not be used when the values of the geometric parameters go beyond the specified ranges.

The approach by Zhang et al (2004) is suitable to estimate the magnitude of lateral displacements associated with liquefaction-induced lateral spread for gently sloping (or level) ground with or without a free face for low to medium-risk projects, or to provide preliminary estimates for higher risk projects. Given the complexity of liquefaction-induced lateral spreads, considerable variations in magnitude and distribution of lateral displacements are expected. Generally, the calculated lateral displacements using the proposed approach for the available case histories showed variations between 50% and 200% of measured values. The accuracy of "measured" lateral displacements for most case histories is about  $\pm 0.1$  to  $\pm 1.92$  m. Therefore, it is unrealistic to expect the accuracy of estimated lateral displacements be within  $\pm 0.1$  m. The reliability of the proposed approach can be fully evaluated only over time with more available case histories.

The approach by Zhang et al (2004) was developed using case history data with limited ranges of earthquake parameters, soil properties, and geometric parameters. Therefore, it is not recommended that the approach be applied for values of input parameters beyond the specified ranges. Engineering judgement and caution should be always exercised in applying the proposed approach and in interpreting the results. Additional new data are required to further evaluate and update the proposed approach.

### Flow Liquefaction

When a soil is strain softening there is the potential for instability. The key response parameter required to estimate if a flow slide would occur is the minimum (residual) undrained shear strength,  $s_{min}$  following strain softening. Methods have been suggested

to estimate the minimum undrained (residual) shear strength of clean sands from penetration resistance based on case histories (Seed and Harder, 1990; Stark and Mesri (1992). A recent re-evaluation of these case histories (Wride et al., 1998) has questioned the validity of the proposed correlation. Recent research has also shown the importance of direction of loading on the minimum undrained shear strength of sands. The minimum undrained shear strength in triaxial compression (TC) loading is higher than that in simple shear (SS), which in turn, is larger than that in triaxial extension (TE). The difference is less as the sand becomes looser. Hence, the appropriate undrained shear strength to be used in a stability analyses will be a function of direction of loading. This has been recognized for some time in clay soils (Bjerrum, 1972).

The possibility of instability in undrained shear is also linked to the brittleness or sensitivity of the soil. Brittleness index (Bishop, 1967) is an index of the collapsibility of a strain softening soil when sheared undrained, which is defined as follows:

$$[15] \quad I_B = \frac{S_{\text{peak}} - S_{\text{min}}}{S_{\text{peak}}}$$

Where:

$S_{\text{peak}}$  = the peak shear resistance prior to strain softening  
 $S_{\text{min}}$  = the minimum undrained shear strength

A value of  $I_B = 1$  indicates zero minimum undrained shear strength. If there is no strain softening then  $I_B = 0$ .

Figure 11 shows a link between the response characteristics of brittleness and minimum undrained strength ratio with only a small influence of direction of loading. When the minimum undrained strength ratio decreases, the brittleness increases. The sands are essentially non-brittle ( $I_B = 0$ ) when the minimum undrained strength ratio ( $S_{\text{min}}/p'$ ) is greater than about 0.2 for TE, 0.25 for SS and 0.40 for TC. These values are similar to those observed for fine grained (clay) soils of low plasticity (Jamiolkowski et al., 1985). When the undrained strength ratio is less than about 0.1 the brittleness is usually high.

Yoshimine et al. (1998) proposed a relationship between normalized cone penetration resistance and the minimum undrained shear strength in simple shear loading for clean sands based on a combination of laboratory testing and case histories, as shown in Figure 12. The method suggested by Robertson and Wride (1998), can be used to calculate the equivalent clean sand normalized penetration resistance,  $(q_{c1N})_{cs}$ , which can then be used to estimate the minimum undrained shear strength ratio using Figure 12. The resulting values of minimum undrained shear strength ratio are approximate and apply primarily to young, normally consolidated, uncemented soils. Sandy soils with angular grains and aged soils would likely have higher strengths. The actual value of  $S_{\text{min}}$  to be applied to any given problem will depend on the ground geometry, but in general, the simple shear direction of loading,  $(S_{\text{min}})_{ss}$ , represents a reasonable average value for most problems.

Soils that have a minimum undrained shear strength ratio in simple shear of around 0.30 or higher are generally not brittle. Soils that have a minimum undrained shear strength ratio of around 0.10 or less are often highly brittle. Hence, the value of  $s_{u(min)}/\sigma'_{vo} = 0.10$  represents the approximate boundary between soils that can show significant strain softening in undrained simple shear and soils that are general not strain softening. For simple shear direction of loading, this boundary can be represented by a normalized clean sand equivalent CPT value of  $(q_{c1N})_{cs} = 50$  (see Figure 12). Hence, a value of  $(q_{c1N})_{cs} < 50$  can be used as an approximate criteria to estimate if a soil may be brittle and strain softening in undrained simple shear. If the soil is cohesionless (i.e. non-plastic,  $I_c < 2.6$ ) the loss of strength could occur rapidly over small strains (brittle). If the soil is cohesive the loss of strength could occur more slowly, depending on the degree of sensitivity and plasticity of the soil. Highly sensitive clays (quick clays) can lose their strength rapidly resulting in flow slides. Less sensitive clays tend to deform gradually without a flow slide.

This method is approximate and tends to produce conservative low values for clayey soils with  $I_c > 2.6$ . This method is appropriate for young uncemented, normally consolidated soils where the vertical effective stress is less than about 250 kPa and  $I_c \leq 2.6$ . In sandy soils where the sand grains are angular the method will under predict the undrained shear strength. Where the in-situ effective stress is greater than 250 kPa, the actual minimum undrained shear strength could be significantly smaller. Soils that fall in the lower left region of the CPT soil behavior chart ( $F < 1\%$ ,  $I_c > 2.6$ ) may be sensitive and may be susceptible to both cyclic and flow liquefaction depending on soil plasticity, sensitivity (brittleness). Clay soils, where  $I_c > 2.6$  and  $F > 1\%$ , are generally non-liquefiable. For high-risk projects, it is advisable to obtain samples to evaluate their liquefaction (strain softening) potential using other criteria.

### Representative Values

When evaluating the potential for flow liquefaction, there is little guidance given on what value of penetration resistance can be taken as representative of the deposit. In the SPT and CPT based methods for cyclic liquefaction suggested by the NCEER Workshop (Youd et al., 2000), the average values were taken from the case histories to develop the method. However, the CPT-based approach is generally applied using all the continuous CPT data. In general, if all values of the measured penetration resistance are used with a relationship that was based on average values, the resulting design will generally be somewhat conservative.

Seed and Harder (1990) and Stark and Mesri (1992) generally used average values from case histories to develop the relationship between  $(N_1)_{60}$  and minimum undrained shear strength to estimate the potential for flow liquefaction. Fear and McRoberts (1995) argued that the minimum value of  $(N_1)_{60}$  would be more appropriate. A disadvantage of defining a criteria based on minimum values is the uncertainty that the measured values represent the minimum. In practice, a lower bound relationship is often applied to all measured penetration resistance values. Recently Popescue et al. (1997) suggested that the 20-percentile value would be appropriate as the representative value for liquefaction.

The 20-percentile value is defined as the value at which 20 percent of the measured values are smaller (i.e. 80 percent are larger). In the authors' opinion, the 20-percentile value is likely the more representative value for any given deposit for the evaluation of flow liquefaction potential.

## Summary

For low-risk, small-scale projects, the potential for cyclic liquefaction can be estimated using penetration tests such as the CPT. The CPT is generally more repeatable than the SPT and is the preferred test, where possible. The CPT provides continuous profiles of penetration resistance, which are useful for identifying soil stratigraphy and for providing continuous profiles of estimated cyclic resistance ratio (CRR). Corrections are required to both the SPT and CPT results for grain characteristics, such as fines content and plasticity. For the CPT, these corrections are best expressed as a function of soil behavior type,  $I_c$ , which is affected by a variety of grain characteristics.

For medium- to high-risk projects, the CPT can be useful for providing a preliminary estimate of liquefaction potential in sandy soils. For higher risk projects, and in regions where there is little previous CPT experience, it is also preferred practice to drill sufficient boreholes adjacent to the CPT soundings to verify various soil types encountered and to perform index testing on disturbed samples. A procedure has been described to correct the measured cone resistance for grain characteristics based on the CPT soil behavior type,  $I_c$ . The corrections are approximate, since the CPT responds to many factors affecting soil behavior. Expressing the corrections in terms of soil behavior index is the preferred method of incorporating the effects of various grain characteristics, more than just fines content. When possible, it is recommended that the corrections be evaluated and modified to suit a specific site and project. However, for small-scale, low-risk projects and in the initial screening process for higher risk projects, the suggested general corrections provide a useful guide and provide continuous profiles that capture the full detail of the soil profile.

It is also useful to evaluate CRR using more than one method. For example, the seismic CPT (SCPT) can provide a useful technique to independently evaluate liquefaction potential, since it measures both the usual CPT parameters, as well as shear wave velocity, within the same soil profile. The CPT provides detailed profiles of cone resistance, but the penetration resistance is sensitive to grain characteristics, such as fines content, soil plasticity and mineralogy, and hence corrections are required. The seismic part of the SCPT provides a shear wave velocity profile typically averaged over 1m intervals and, therefore contains less detail than the cone tip resistance profile. However, shear wave velocity is less influenced by grain characteristics and few or no corrections are required (Andrus and Stokoe, 1998). Shear wave velocity should be measured with care to provide the most accurate results, since the estimated CRR is sensitive to small changes in shear wave velocity. There should be consistency in the liquefaction evaluation using either method. If the two methods provide different predictions of CRR profiles, samples should be obtained to evaluate the grain characteristics of the soil.

A key advantage of the integrated CPT method is that the algorithms can easily be incorporated into a spreadsheet or software, as illustrated by the Moss Landing example. The result is a straightforward method for analysing the entire CPT profile in a continuous manner. This provides a useful tool for the engineer to review the potential for cyclic liquefaction across a site using engineering judgement.

An extension to the CPT-based method to evaluate liquefaction potential was suggested by Zhang et al (2002 and 2004) and allows estimates of vertical settlements and lateral spread deformations to be made using the continuous CPT profile.

The method by Zhang et al (2002) to estimate post-earthquake vertical settlements was evaluated using liquefaction-induced ground settlements from the Marina District and Treasure Island case histories damaged by liquefaction during the 1989 Loma Prieta earthquake. Good agreement between the calculated and measured liquefaction-induced ground settlements was found. Although further evaluations of the method are required with future case history data from different earthquakes and ground conditions, as they become available, it is suggested that the CPT-based method may be used to estimate liquefaction-induced settlements for low to medium risk projects and also provide preliminary estimates for higher risk projects.

The method by Zhang et al (2004) to estimate lateral displacements was developed using case history data with limited ranges of earthquake parameters, soil properties, and geometric parameters. Therefore, it is not recommended that the approach be applied for values of input parameters beyond the specified ranges. Engineering judgement and caution should be always exercised in applying the proposed approach and in interpreting the results. Additional new data are required to further evaluate and update the proposed approach.

Soils that have a minimum undrained shear strength ratio of around 0.10 or less are often highly brittle. Hence, the value of  $s_{u(min)}/\sigma'_{vo} = 0.10$  represents the approximate boundary between soils that can show significant strain softening in undrained simple shear and soils that are general not strain softening. Research has clearly shown that the undrained shear strength of soils is usually a function of direction of loading, with undrained shear strengths in compression loading often being higher than those in simple shear and triaxial extension. The resulting average minimum undrained shear strength is therefore a function of the slope geometry. Although all projects should be evaluated based on their actual geometry, often the average undrained shear strength is close to that in simple shear loading, which is consistent with the observations made by Bjerrum (1972) for slopes and embankments in clays. For simple shear direction of loading, this boundary can be represented by a normalized clean sand equivalent CPT value of  $(q_{c1N})_{cs} = 50$ . Hence, a value of  $(q_{c1N})_{cs} < 50$  can be used as an approximate criteria to estimate if a soil may be brittle and strain softening in undrained simple shear. If the soil is cohesionless (i.e. non-plastic,  $I_c < 2.6$ ) the loss of strength could occur rapidly over small strains (brittle). If the soil is cohesive the loss of strength could occur more slowly, depending on the degree of sensitivity and plasticity of the soil. Highly sensitive clays (quick clays) can loose their strength rapidly resulting in flow slides. Less sensitive clays tend to deform gradually without a flow slide.

Sands that have angular grains may have a minimum undrained strength ratio higher than predicted using the suggested CPT chart. Aged soils (age > 1,000 years) may also be somewhat stronger. For high-risk projects, the proposed CPT method provides a useful screening technique to identify potentially critical zones where flow liquefaction may be possible. For low risk projects, the proposed CPT method will generally provide a conservative estimate of the minimum undrained shear strength ratio in simple shear loading. The proposed relationship conservatively estimates the minimum undrained shear strength ratio in simple shear for a soil structure that contains extensive amounts of loose soils with impeded drainage, such as thick deposits of loose interbedded sands and silts. In soil structures where drainage and consolidation of the liquefied layer can occur during and immediately after the earthquake, higher values of undrained shear strength will likely exist. Such conditions may exist in a thin deposit with free drainage to the ground surface or a deposit interbedded with extensive pervious gravel layers.

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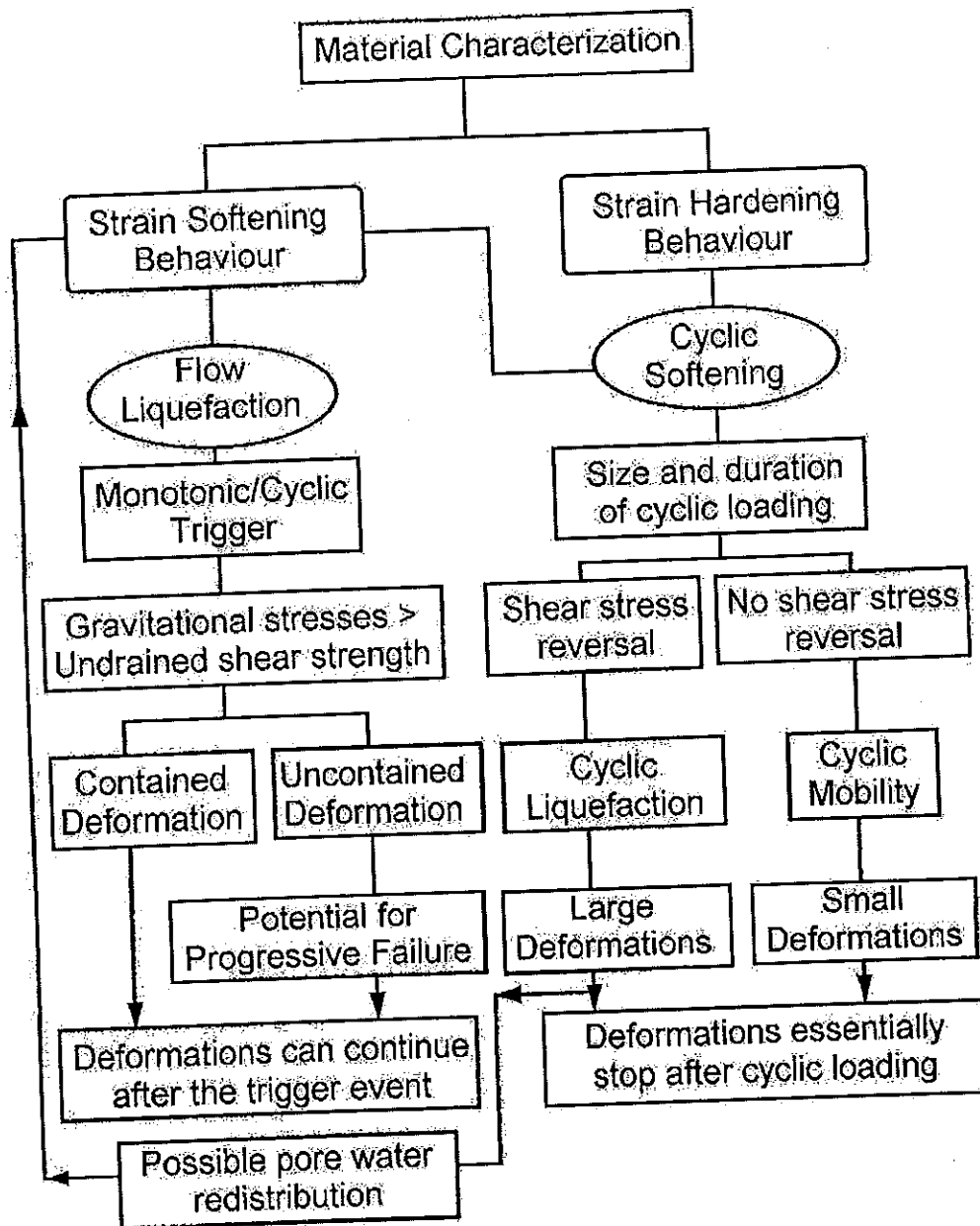


Figure 1. Flow chart to evaluate liquefaction of soils  
(After Robertson and Wride, 1998)

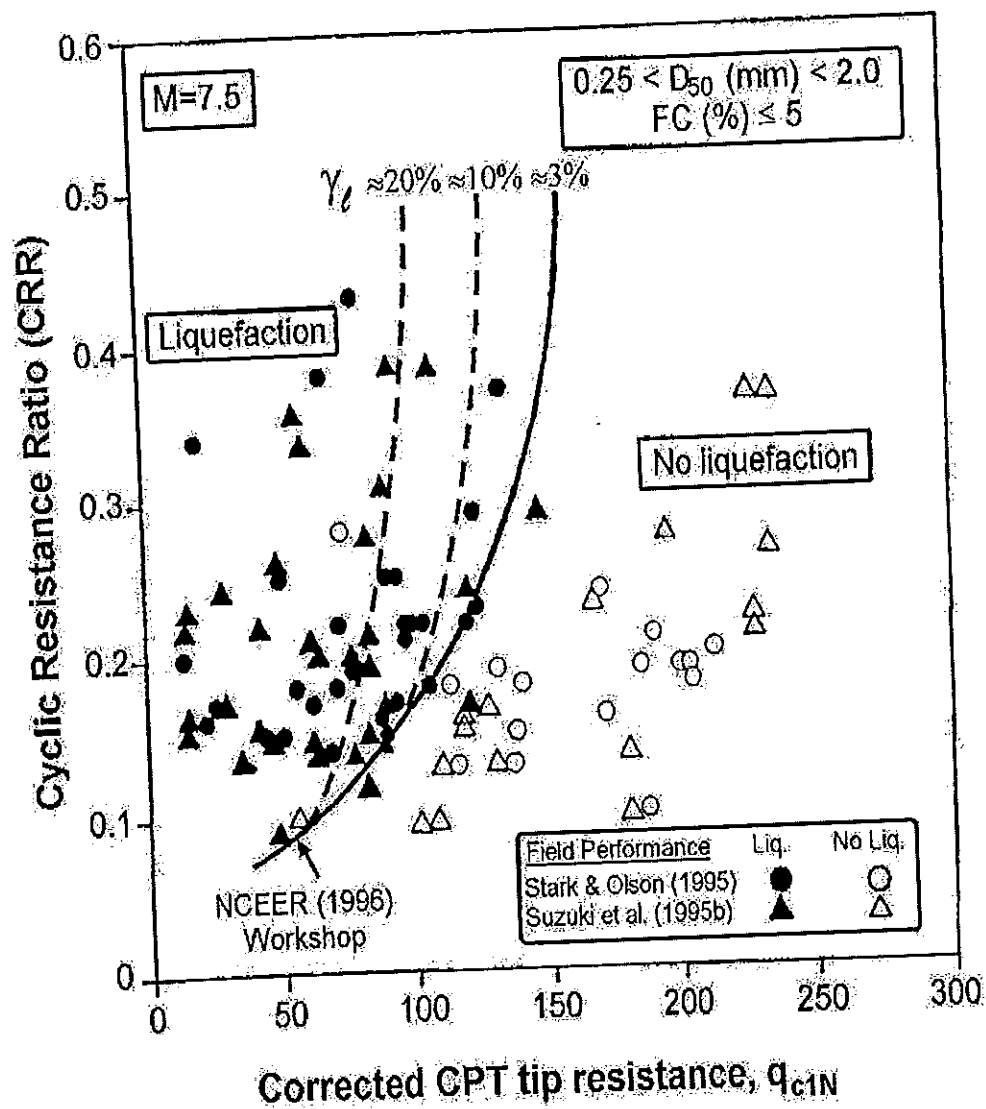
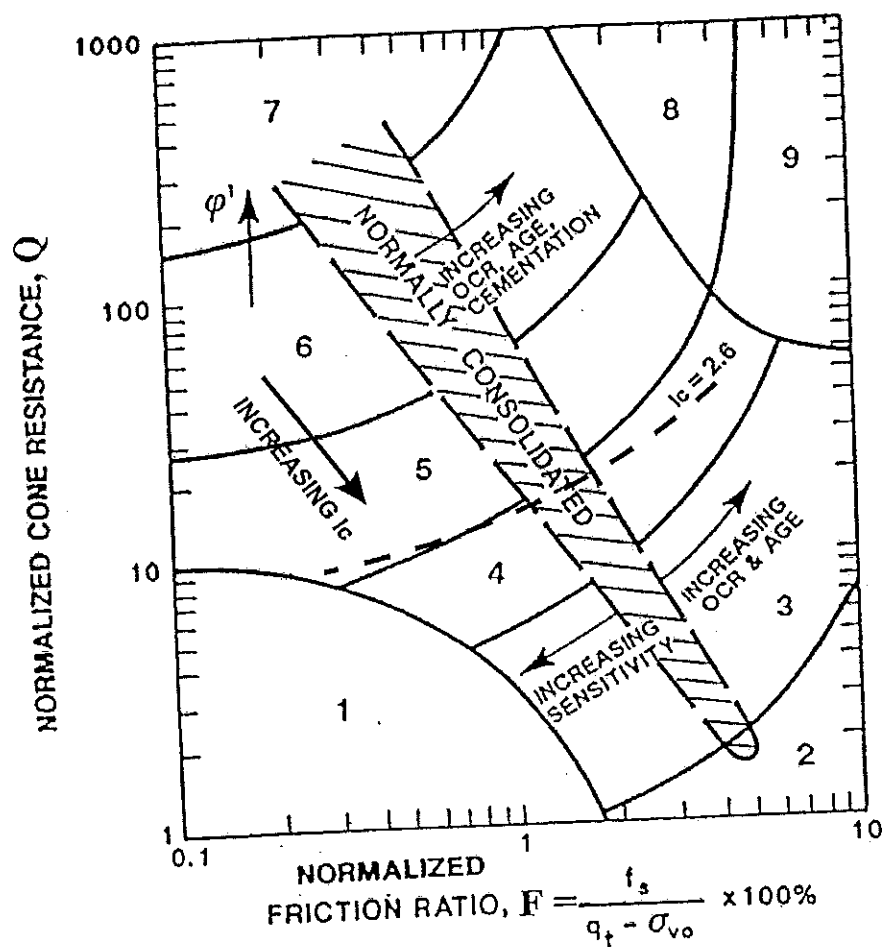


Figure 2. Cyclic resistance ratio (CRR) from the CPT for clean sands. (After Robertson and Wride, 1997).



Zone	Soil Behaviour Type	$I_c$
1	Sensitive, fine grained	N/A
2	Organic soils – peats	> 3.6
3	Clays – silty clay to clay	2.95 – 3.6
4	Silt mixtures – clayey silt to silty clay	2.60 – 2.95
5	Sand mixtures – silty sand to sandy silt	2.05 – 2.6
6	Sands – clean sand to silty sand	1.31 – 2.05
7	Gravelly sand to dense sand	< 1.31
8	Very stiff sand to clayey sand*	N/A
9	Very stiff, fine grained*	N/A

\* Heavily overconsolidated or cemented

Note: Soil behaviour type index ( $I_p$ ) is given by  
 $I_c = [(3.47 - \log Q)^2 + (\log F + 1.22)^2]^{0.5}$

Figure 3. Normalized CPT soil behaviour type chart. (After Robertson, 1990).

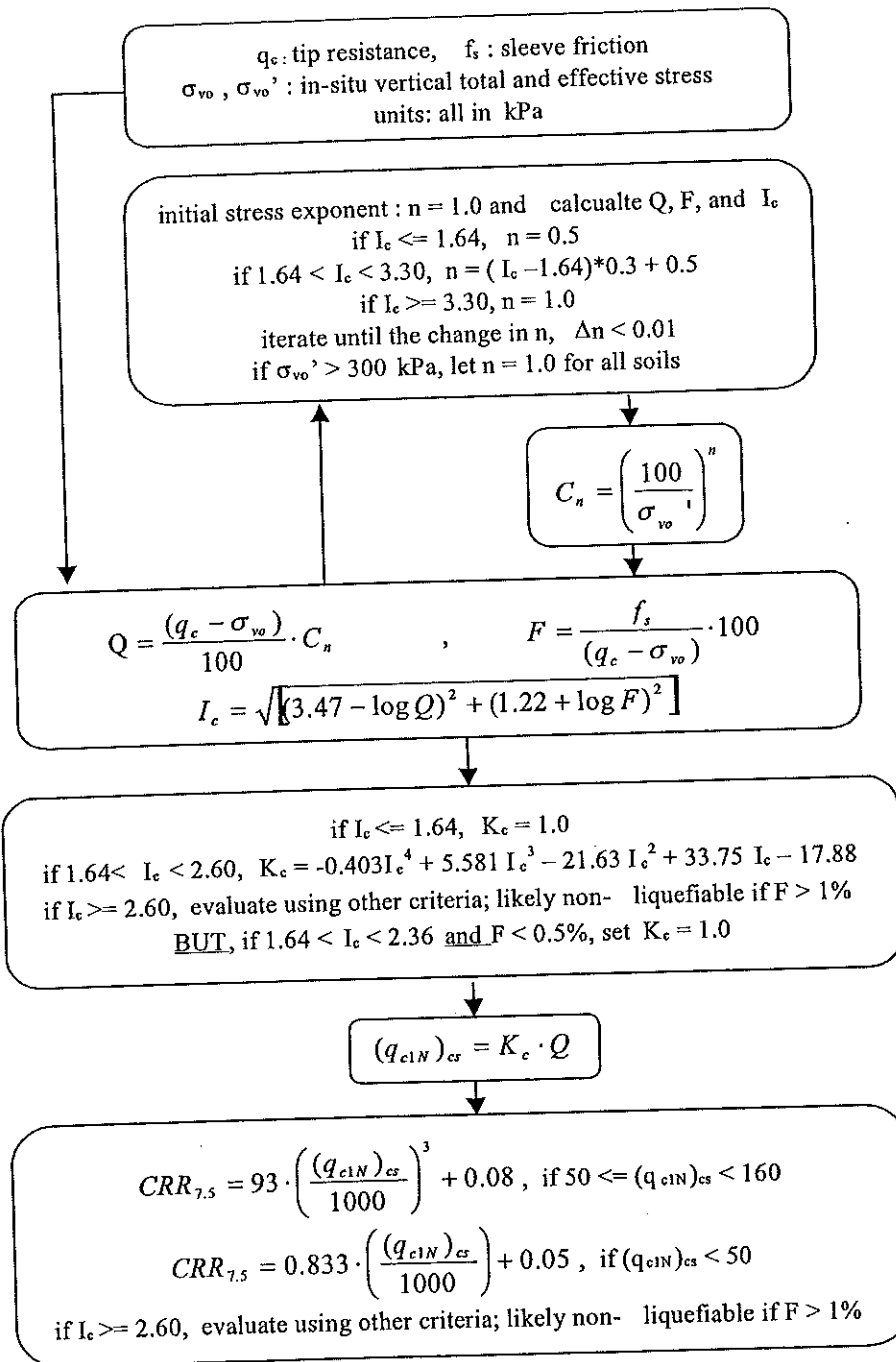


Figure 4. Flow chart to evaluate cyclic resistance ratio (CRR) from the CPT. (Modified from Robertson and Wride, 1997).

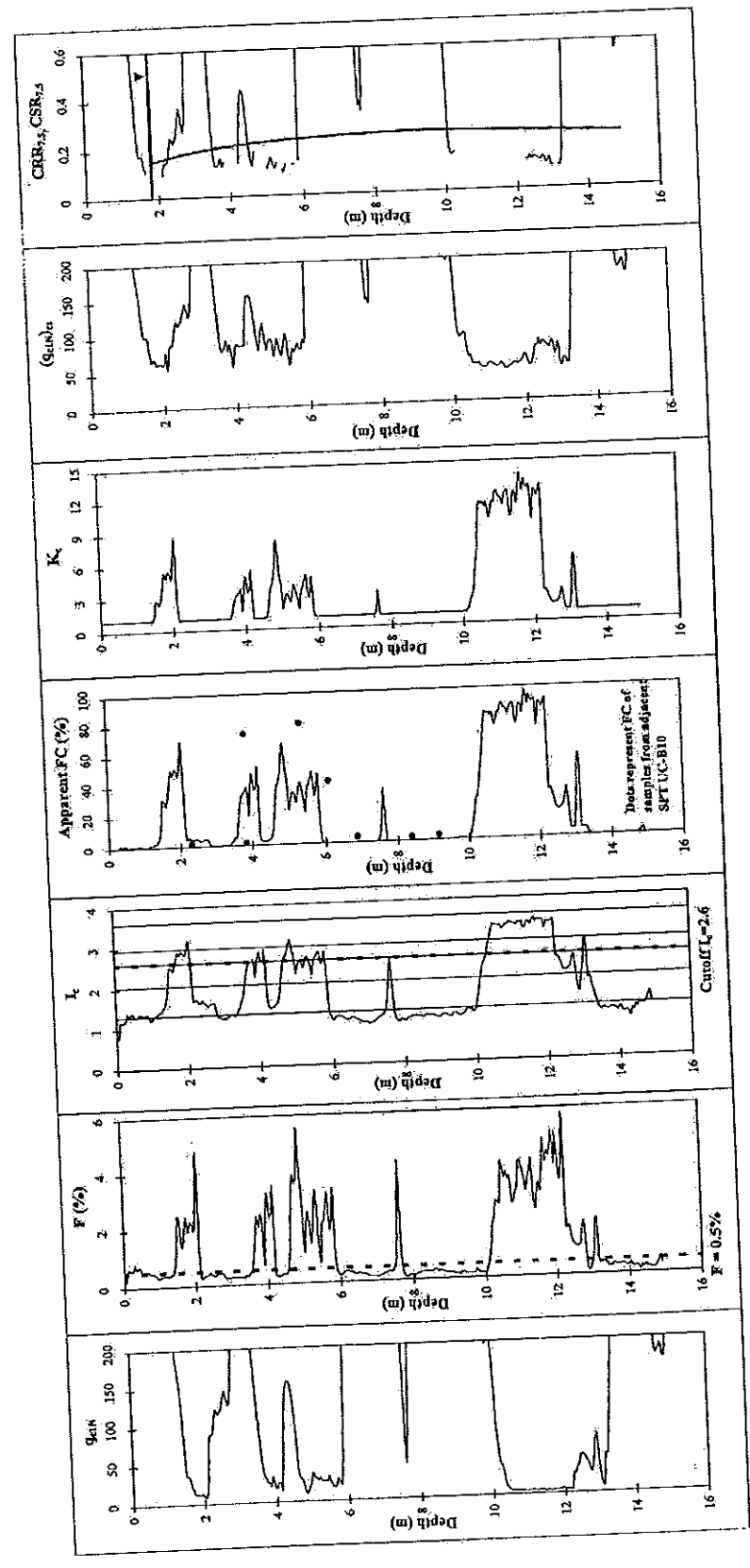


Figure 5. Example of CPT to evaluate cyclic liquefaction at Moss Landing Site. (After Robertson and Wride, 1998).

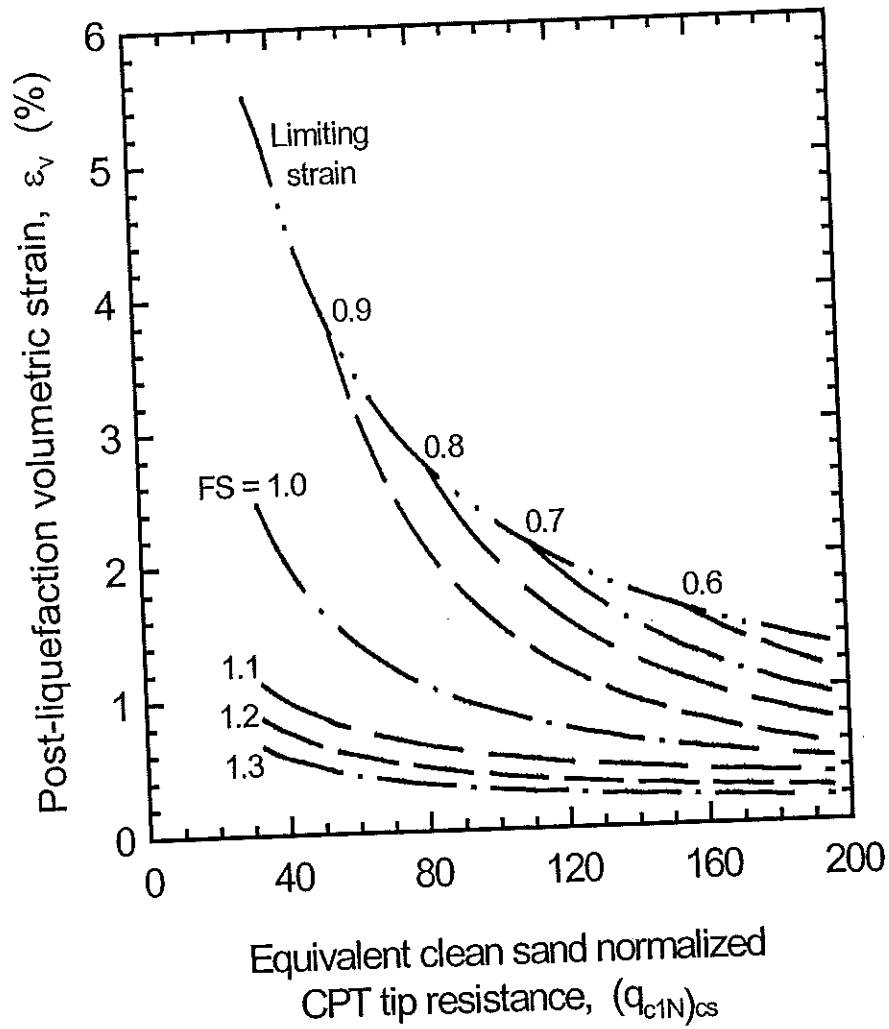


Figure 6. Relationship between post-liquefaction volumetric strain and equivalent clean sand normalized CPT tip resistance for different factors of safety (FS). (After Zhang et al., 2002)

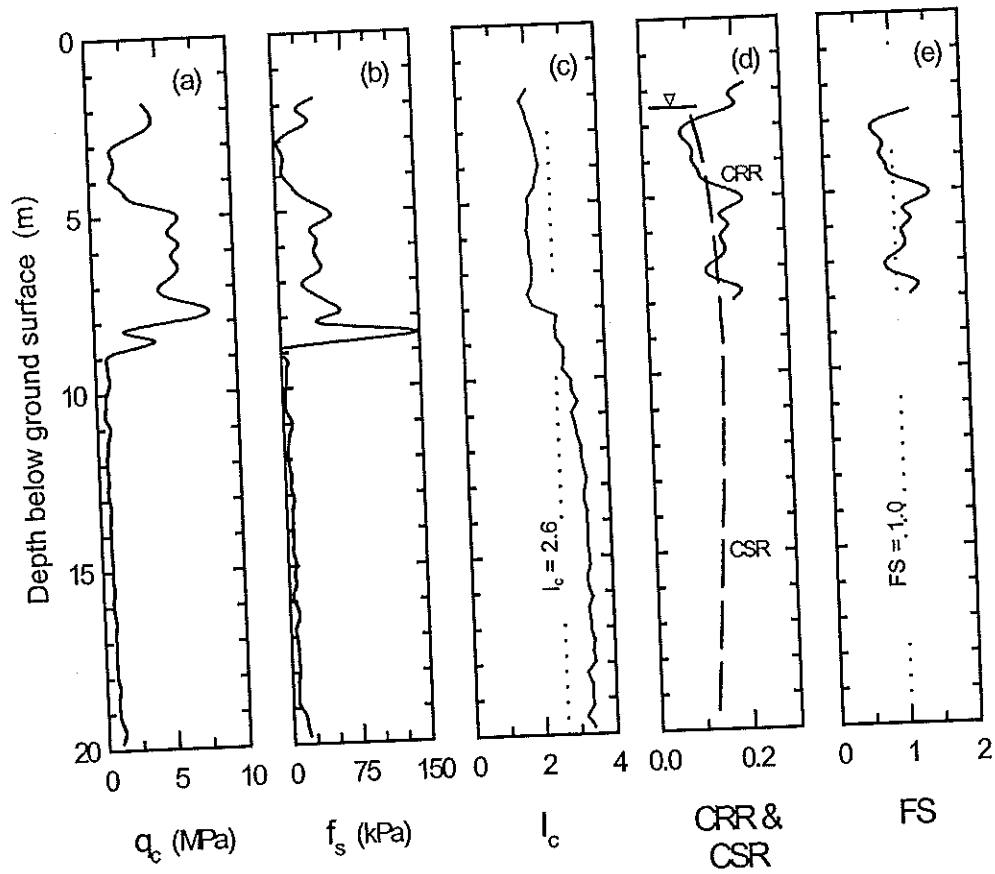


Figure 7. Example plots illustrating the major procedures in performing liquefaction potential analysis using the CPT based Robertson and Wride's (1998) method. (After Zhang et al., 2002)

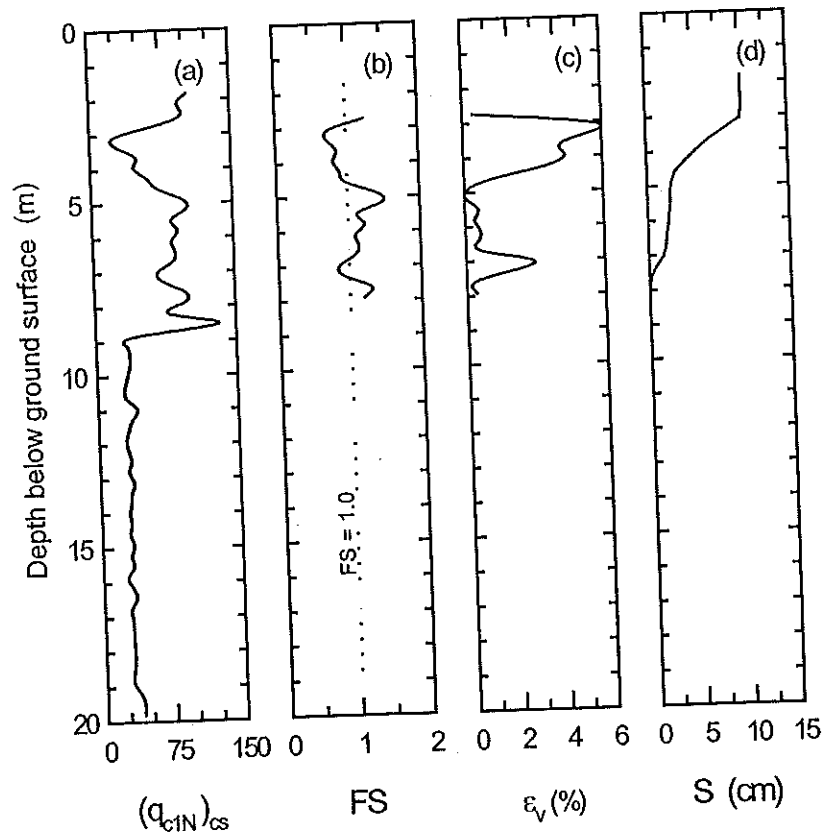


Figure 8. Example plots illustrating the major procedures in estimating liquefaction-induced ground settlements using the Zhang et al (2002) CPT based method.



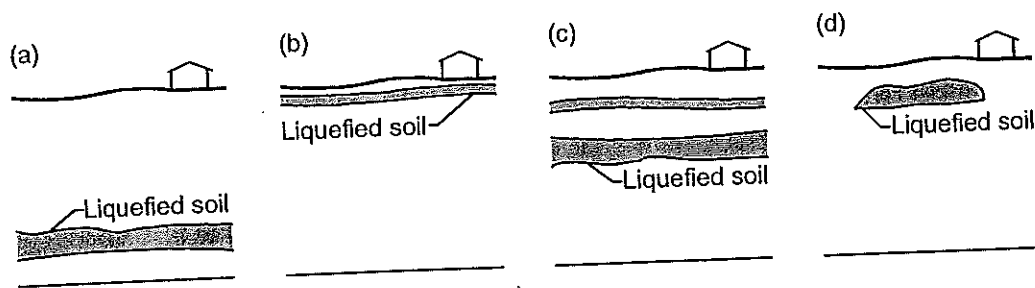


Figure 9. Four hypothetical cases showing importance of three-dimensional distribution of liquefied layers.

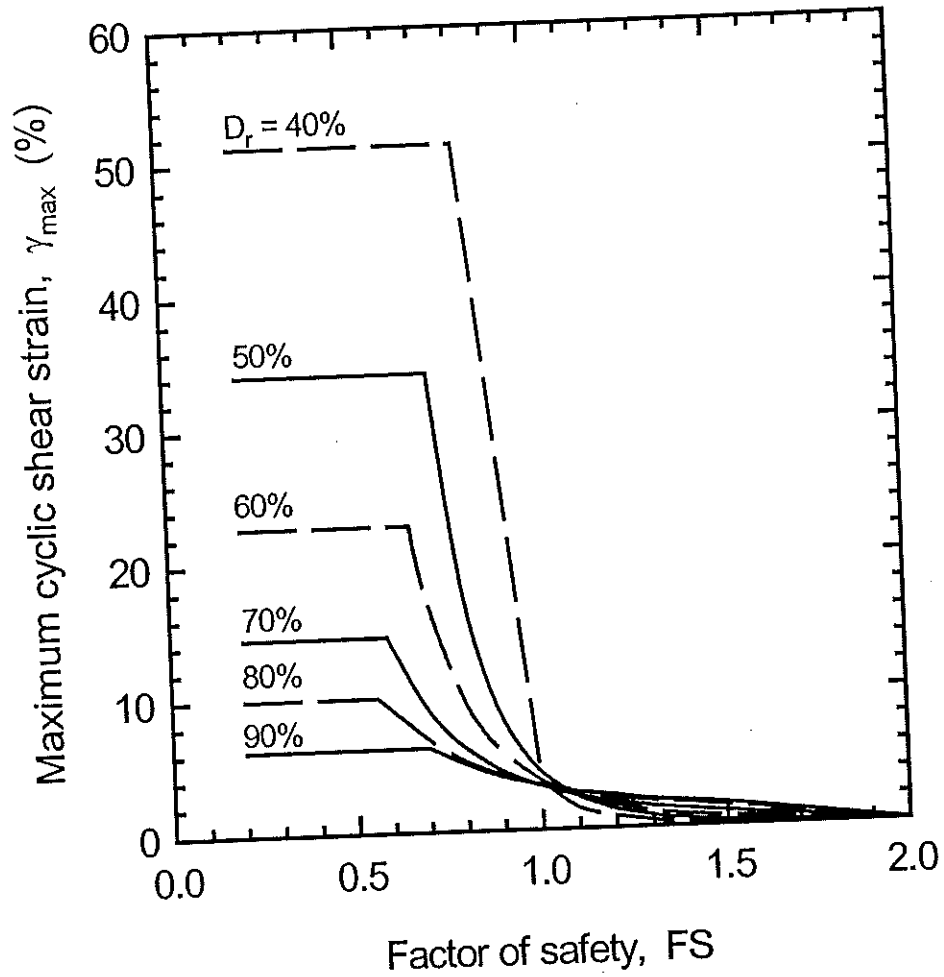


Figure 10. Relationship between maximum cyclic shear strain and factor of safety for different relative densities  $D_r$  for clean sands (After Zhang et al., 2004)

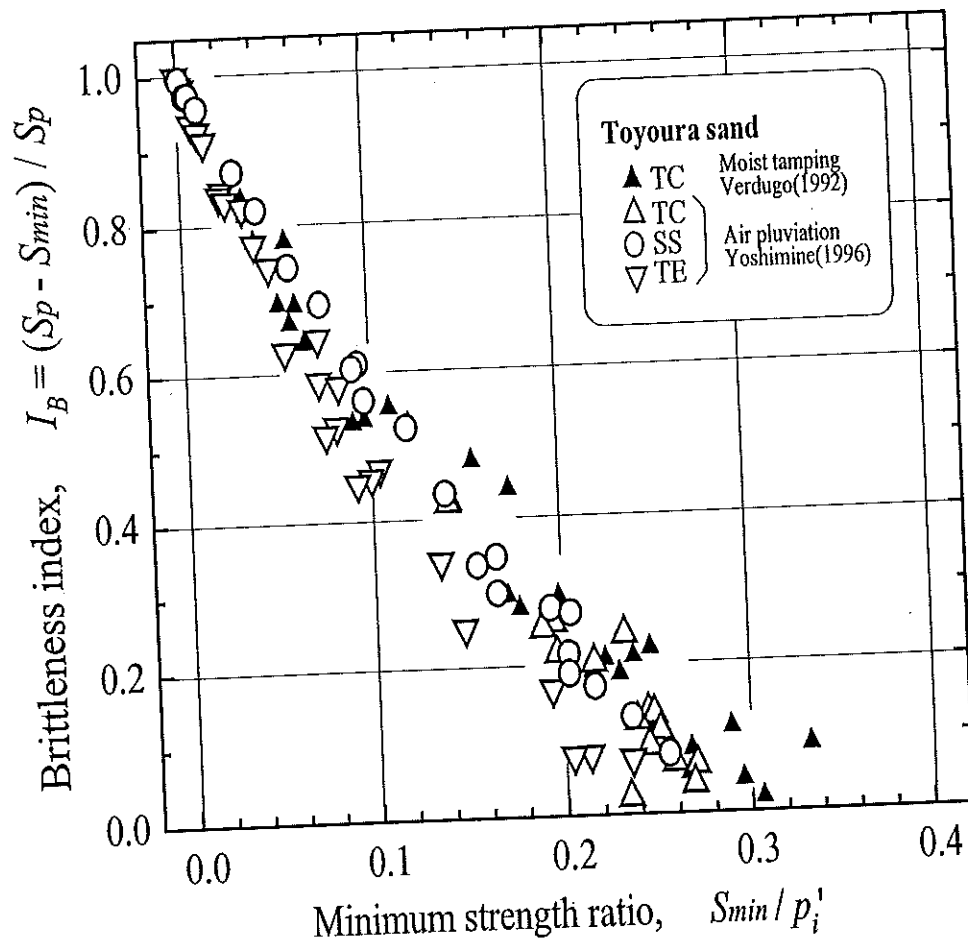


Figure 11. Relationship between minimum undrained strength ratio and brittleness index for clean sand (Yoshimine et al., 1998)

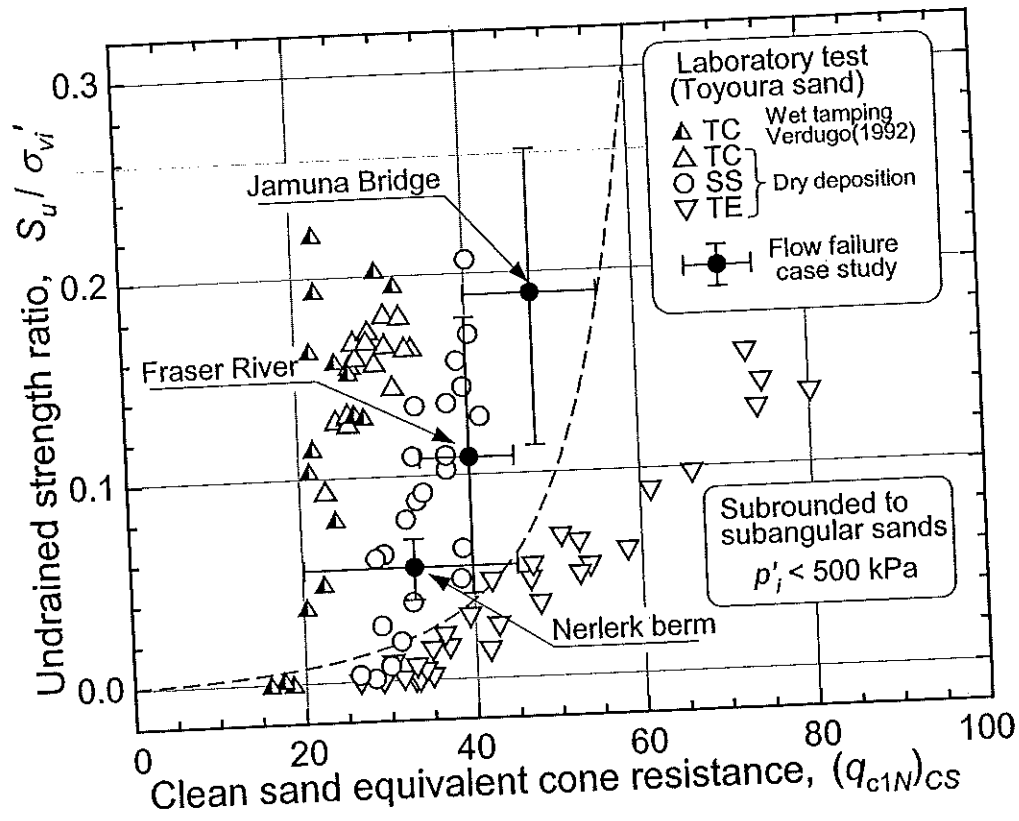


Figure 12. Undrained strength ratio from CPT for clean sand. (After Yoshimine et al., 1998).

## Appendix B

### Application of $V_s$ for Estimating Liquefaction Parameters



# SHEAR WAVE VELOCITY METHOD FOR ESTIMATING LIQUEFACTION PARAMETERS

(NCEER Workshop, 1998)

Hole

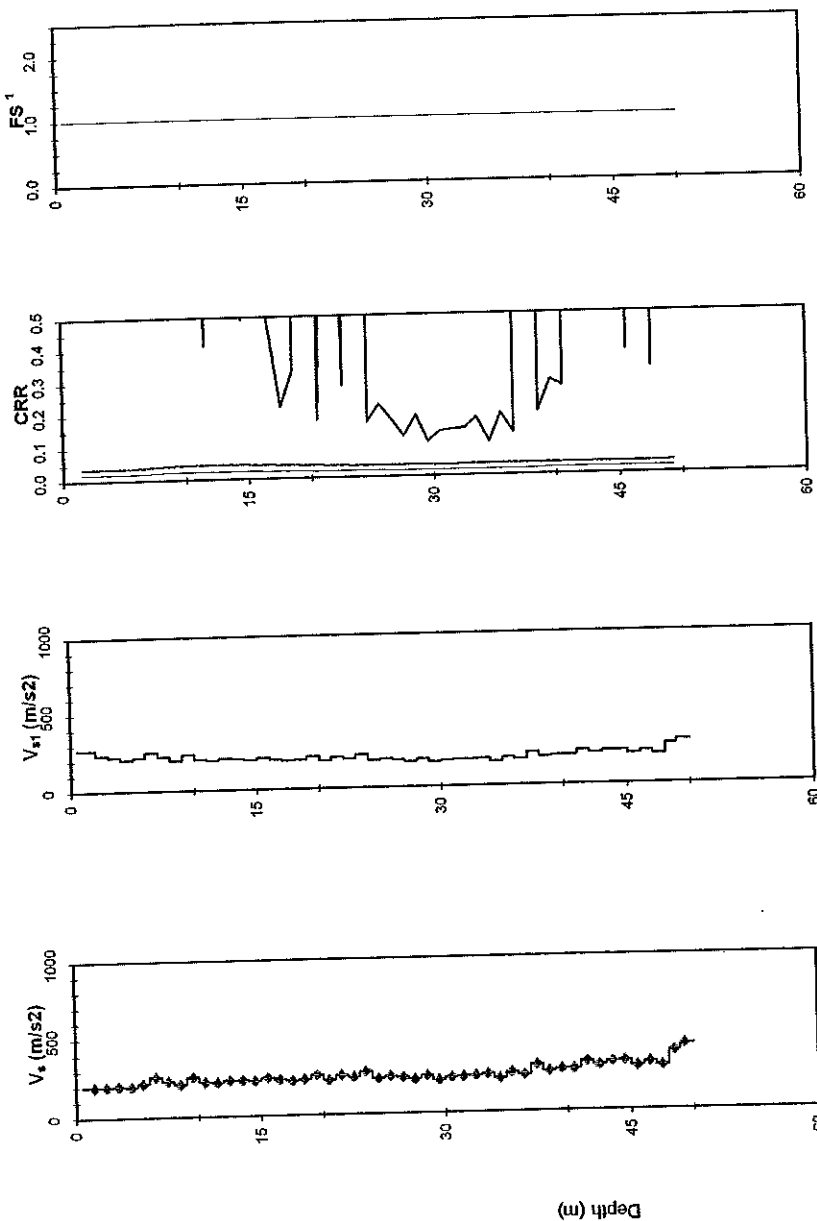
05:17:04 12:50 CONETEC

CPT: Location: HWY69 4L ESTAIRE 20 Ton St 113 SCPT04-01S

WT: 5.70m

Date:

Depth: 49.25m



CSR mag = 6.00  
max a = 0.05  
max a = 0.1

FS = 1.0  
max a = 0.05  
max a = 0.1

<sup>1</sup> Empirical data indicate no observed liquefaction below 20m (65ft).

**SHEAR WAVE VELOCITY METHOD FOR EVALUATING CYCLIC RESISTANCE RATIO (CRR)**  
(NCEER Workshop, 1998)

File Data  
File Name  
Location  
Cone

Date  
20 Ton St 11: Time  
Operator

ETEC

Site Parameters

Water Table  
Unit Weight of Water  
Atmospheric Pressure

Earthquake Parameters

Earthquake Magnitude (typ 7.5)  
max acceleration 1  
max acceleration 2  
Maximum shear wave velocity

Depth (m)	$V_s$ (m/s <sup>2</sup> )	$\gamma$ (kN/m <sup>3</sup> )	$\sigma_{v'}$ (kN/m <sup>2</sup> )	$\sigma_{v'}$ (kN/m <sup>2</sup> )	$V_{s1}$ (m/s <sup>2</sup> )	CRR	CSR <sub>7.5</sub> a max 1	CSR <sub>7.5</sub> a max 2	CSR <sub>vr</sub> a max 1	CSR <sub>vr</sub> a max 2	FS a max 1	FS a max 2
1.50	195.00	18.50	27.75	27.75	268.67	1000.00	0.03	0.06	0.02	0.04	1000.00	1000.00
2.50	193.95	18.50	46.25	46.25	235.18	1000.00	0.03	0.06	0.02	0.04	1000.00	1000.00
3.50	199.71	18.50	64.75	64.75	222.63	1000.00	0.03	0.06	0.02	0.04	1000.00	1000.00
4.50	194.58	18.50	83.25	83.25	203.70	1000.00	0.03	0.06	0.02	0.04	1000.00	1000.00
5.50	217.03	18.50	101.75	101.75	216.09	1000.00	0.03	0.06	0.02	0.04	1000.00	1000.00
6.50	255.71	18.50	120.25	120.25	248.34	1000.00	0.03	0.07	0.02	0.04	1000.00	1000.00
7.50	229.94	18.50	138.75	138.75	219.20	1000.00	0.04	0.07	0.02	0.04	1000.00	1000.00
8.50	209.31	18.50	157.25	157.25	196.10	0.79	0.04	0.07	0.02	0.04	37.76	18.88
9.50	254.78	18.50	175.75	175.75	234.86	1000.00	0.04	0.08	0.02	0.04	1000.00	1000.00
10.50	219.07	18.50	194.25	194.25	198.90	2.62	0.04	0.08	0.02	0.04	120.20	60.10
11.50	214.41	18.50	212.75	212.75	191.89	0.41	0.04	0.08	0.02	0.04	18.89	9.44
12.50	229.72	18.50	231.25	231.25	202.82	1000.00	0.04	0.08	0.02	0.04	1000.00	1000.00
13.50	227.13	18.50	249.75	249.75	197.98	1.46	0.04	0.08	0.02	0.04	22.98	11.49
14.50	224.60	18.50	268.25	268.25	193.38	0.49	0.04	0.07	0.02	0.04	1000.00	1000.00
15.50	242.01	18.50	286.75	286.75	205.97	1000.00	0.04	0.07	0.02	0.04	23.76	11.88
16.50	229.79	18.50	305.25	305.25	193.40	0.49	0.04	0.07	0.02	0.04	10.83	5.41
17.50	219.22	18.50	323.75	323.75	182.54	0.22	0.04	0.07	0.02	0.04	16.62	8.31
18.50	229.81	18.50	342.25	342.25	189.41	0.33	0.04	0.07	0.02	0.04	1000.00	1000.00
19.50	255.02	18.50	360.75	360.75	208.14	1000.00	0.03	0.07	0.02	0.04	9.51	4.76
20.50	219.24	18.50	379.25	379.25	177.25	0.18	0.03	0.07	0.02	0.04	127.05	63.53
21.50	248.06	18.50	397.75	397.75	198.74	2.30	0.03	0.06	0.02	0.04	16.15	8.08
22.50	235.80	18.50	416.25	416.25	187.25	0.28	0.03	0.06	0.02	0.03	1000.00	1000.00
23.50	269.49	18.50	434.75	434.75	212.20	1000.00	0.03	0.06	0.02	0.03	9.85	4.92
24.50	224.68	18.50	453.25	453.25	175.46	0.17	0.03	0.06	0.02	0.03	13.07	6.53
25.50	235.81	18.50	471.75	471.75	182.70	0.22	0.03	0.06	0.02	0.03	10.41	5.20
26.50	229.85	18.50	490.25	490.25	176.71	0.17	0.03	0.06	0.02	0.03	7.33	3.66
27.50	214.56	18.50	508.75	508.75	163.73	0.12	0.03	0.06	0.02	0.03	11.33	5.67
28.50	235.82	18.50	527.25	527.25	178.65	0.19	0.03	0.06	0.02	0.03	6.44	3.22
29.50	208.74	18.50	545.75	545.75	157.03	0.11	0.03	0.06	0.02	0.03	8.42	4.21
30.50	225.20	18.50	564.25	564.25	168.25	0.14	0.03	0.06	0.02	0.03	8.63	4.31
31.50	228.28	18.50	582.75	582.75	169.42	0.14	0.03	0.06	0.02	0.03	8.78	4.39
32.50	230.92	18.50	601.25	601.25	170.27	0.14	0.03	0.06	0.02	0.03	10.71	5.35
33.50	241.52	18.50	619.75	619.75	176.95	0.18	0.03	0.06	0.02	0.03	6.14	3.07
34.50	213.20	18.50	638.25	638.25	155.24	0.10	0.03	0.06	0.02	0.03		

Depth (m)	$V_s$ (m/s <sup>2</sup> )	$\gamma$ (kN/m <sup>3</sup> )	$G_{cr}$ (kN/m <sup>2</sup> )	$C_{10}$ (kN/m <sup>2</sup> )	$V_{s1}$ (m/s <sup>2</sup> )	CRR	CSR <sub>7.5</sub> a max 1	CSR <sub>7.5</sub> a max 2	CSR <sub>var</sub> a max 1	CSR <sub>var</sub> a max 2	FS a max 1	FS a max 2
35.50	246.89	18.50	656.75	364.41	178.69	0.19	0.03	0.06	0.02	0.03	11.29	5.64
36.50	230.93	18.50	675.25	373.10	166.16	0.13	0.03	0.06	0.02	0.03	7.76	3.88
37.50	294.09	18.50	693.75	381.79	210.39	1000.00	0.03	0.06	0.02	0.03	1000.00	1000.00
38.50	252.51	18.50	712.25	390.48	179.63	0.19	0.03	0.06	0.02	0.03	11.56	5.78
39.50	265.23	18.50	730.75	399.17	187.64	0.29	0.03	0.06	0.02	0.03	17.18	8.59
40.50	265.23	18.50	749.25	407.86	186.64	0.27	0.03	0.06	0.02	0.03	16.06	8.03
41.50	310.54	18.50	767.75	416.55	217.37	1000.00	0.03	0.06	0.02	0.03	1000.00	1000.00
42.50	285.70	18.50	786.25	425.24	198.95	2.74	0.03	0.06	0.02	0.03	160.75	80.37
43.50	305.79	18.50	804.75	433.93	211.87	1000.00	0.03	0.06	0.02	0.03	1000.00	1000.00
44.50	310.54	18.50	823.25	442.62	214.10	1000.00	0.03	0.06	0.02	0.03	1000.00	1000.00
45.50	278.54	18.50	841.75	451.31	191.10	0.36	0.03	0.06	0.02	0.03	22.15	11.07
46.50	302.10	18.50	860.25	460.00	206.28	1000.00	0.03	0.06	0.02	0.03	1000.00	1000.00
47.50	278.54	18.50	878.75	468.69	189.30	0.33	0.03	0.06	0.02	0.03	18.89	9.44
48.50	374.51	18.50	897.25	477.38	253.37	1000.00	0.03	0.06	0.02	0.03	1000.00	1000.00
49.25	416.65	18.50	911.13	483.90	280.92	1000.00	0.03	0.06	0.02	0.03	1000.00	1000.00



# LIQUEFACTION RESISTANCE OF SOILS: SUMMARY REPORT FROM THE 1996 NCEER AND 1998 NCEER/NSF WORKSHOPS ON EVALUATION OF LIQUEFACTION RESISTANCE OF SOILS<sup>a</sup>

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**ABSTRACT:** Following disastrous earthquakes in Alaska and in Niigata, Japan in 1964, Professors H. B. Seed and I. M. Idriss developed and published a methodology termed the "simplified procedure" for evaluating liquefaction resistance of soils. This procedure has become a standard of practice throughout North America and much of the world. The methodology which is largely empirical, has evolved over years, primarily through summary papers by H. B. Seed and his colleagues. No general review or update of the procedure has occurred, however, since 1985, the time of the last major paper by Professor Seed and a report from a National Research Council workshop on liquefaction of soils. In 1996 a workshop sponsored by the National Center for Earthquake Engineering Research (NCEER) was convened by Professors T. L. Youd and I. M. Idriss with 20 experts to review developments over the previous 10 years. The purpose was to gain consensus on updates and augmentations to the simplified procedure. The following topics were reviewed and recommendations developed: (1) criteria based on standard penetration tests; (2) criteria based on cone penetration tests; (3) criteria based on shear-wave velocity measurements; (4) use of the Becker penetration test for gravelly soil; (4) magnitude scaling factors; (5) correction factors for overburden pressures and sloping ground; and (6) input values for earthquake magnitude and peak acceleration. Probabilistic and seismic energy analyses were reviewed but no recommendations were formulated.

## INTRODUCTION

Over the past 25 years a methodology termed the "simplified procedure" has evolved as a standard of practice for evaluating the liquefaction resistance of soils. Following disastrous earthquakes in Alaska and in Niigata, Japan in 1964, Seed and Idriss (1971) developed and published the basic "simplified procedure." That procedure has been modified and improved periodically since that time, primarily through landmark papers by Seed (1979), Seed and Idriss (1982), and Seed et al. (1985). In 1985, Professor Robert V. Whitman convened a workshop on behalf of the National Research Council (NRC) in which 36 experts and observers thoroughly reviewed the state-of-knowledge and the state-of-the-art for assessing liquefaction hazard. That workshop produced a report (NRC 1985) that has become a widely used standard and reference for liquefaction hazard assessment. In January 1996, T. L. Youd and I. M. Idriss convened a workshop of 20 experts to update the simplified procedure and incorporate research findings from the previous decade. This paper summarizes recommendations from that workshop (Youd and Idriss 1997).

To keep the workshop focused, the scope of the workshop was limited to procedures for evaluating liquefaction resistance of soils under level to gently sloping ground. In this context, liquefaction refers to the phenomena of seismic generation of large pore-water pressures and consequent softening of granular soils. Important postliquefaction phenomena, such as residual shear strength, soil deformation, and ground failure, were beyond the scope of the workshop.

The simplified procedure was developed from empirical evaluations of field observations and field and laboratory test data. Field evidence of liquefaction generally consisted of surficial observations of sand boils, ground fissures, or lateral spreads. Data were collected mostly from sites on level to

gently sloping terrain, underlain by Holocene alluvial or fluvial sediment at shallow depths (<15 m). The original procedure was verified for, and is applicable only to, these site conditions. Similar restrictions apply to the implementation of the updated procedures recommended in this report.

Liquefaction is defined as the transformation of a granular material from a solid to a liquefied state as a consequence of increased pore-water pressure and reduced effective stress (Marcuson 1978). Increased pore-water pressure is induced by the tendency of granular materials to compact when subjected to cyclic shear deformations. The change of state occurs most readily in loose to moderately dense granular soils with poor drainage, such as silty sands or sands and gravels capped by or containing seams of impermeable sediment. As liquefaction occurs, the soil stratum softens, allowing large cyclic deformations to occur. In loose materials, the softening is also accompanied by a loss of shear strength that may lead to large shear deformations or even flow failure under moderate to high shear stresses, such as beneath a foundation or sloping ground. In moderately dense to dense materials, liquefaction leads to transient softening and increased cyclic shear strains, but a tendency to dilate during shear inhibits major strength loss and large ground deformations. A condition of cyclic mobility or cyclic liquefaction may develop following liquefaction of moderately dense granular materials. Beneath gently sloping to flat ground, liquefaction may lead to ground oscillation or lateral spread as a consequence of either flow deformation or cyclic mobility. Loose soils also compact during liquefaction and reconsolidation, leading to ground settlement. Sand boils may also erupt as excess pore water pressures dissipate.

## CYCLIC STRESS RATIO (CSR) AND CYCLIC RESISTANCE RATIO (CRR)

Calculation, or estimation, of two variables is required for evaluation of liquefaction resistance of soils: (1) the seismic demand on a soil layer, expressed in terms of CSR; and (2) the capacity of the soil to resist liquefaction, expressed in terms of CRR. The latter variable has been termed the cyclic stress ratio or the cyclic stress ratio required to generate liquefaction, and has been given different symbols by different writers. For example, Seed and Harder (1990) used the symbol  $CSR_L$ , Youd (1993) used the symbol  $CSRL$ , and Kramer

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(1996) used the symbol  $CSR_L$  to denote this ratio. To reduce confusion and to better distinguish induced cyclic shear stresses from mobilized liquefaction resistance, the capacity of a soil to resist liquefaction is termed the CRR in this report. This term is recommended for engineering practice.

## EVALUATION OF CSR

Seed and Idriss (1971) formulated the following equation for calculation of the cyclic stress ratio:

$$CSR = (\tau_{av}/\sigma'_{vo}) = 0.65(a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d \quad (1)$$

where  $a_{max}$  = peak horizontal acceleration at the ground surface generated by the earthquake (discussed later);  $g$  = acceleration of gravity;  $\sigma_{vo}$  and  $\sigma'_{vo}$  are total and effective vertical overburden stresses, respectively; and  $r_d$  = stress reduction coefficient. The latter coefficient accounts for flexibility of the soil profile. The workshop participants recommend the following minor modification to the procedure for calculation of CSR.

For routine practice and noncritical projects, the following equations may be used to estimate average values of  $r_d$  (Liao and Whitman 1986b):

$$r_d = 1.0 - 0.00765z \quad \text{for } z \leq 9.15 \text{ m} \quad (2a)$$

$$r_d = 1.174 - 0.0267z \quad \text{for } 9.15 \text{ m} < z \leq 23 \text{ m} \quad (2b)$$

where  $z$  = depth below ground surface in meters. Some investigators have suggested additional equations for estimating  $r_d$  at greater depths (Robertson and Wride 1998), but evaluation of liquefaction at these greater depths is beyond the depths where the simplified procedure is verified and where routine applications should be applied. Mean values of  $r_d$  calculated from (2) are plotted in Fig. 1, along with the mean and range

of values proposed by Seed and Idriss (1971). The workshop participants agreed that for convenience in programming spreadsheets and other electronic aids, and to be consistent with past practice,  $r_d$  values determined from (2) are suitable for use in routine engineering practice. The user should understand, however, that there is considerable variability in the flexibility and thus  $r_d$  at field sites, that  $r_d$  calculated from (2) are the mean of a wide range of possible  $r_d$ , and that the range of  $r_d$  increases with depth (Golesorkhi 1989).

For ease of computation, T. F. Blake (personal communication, 1996) approximated the mean curve plotted in Fig. 1 by the following equation:

$$r_d = \frac{(1.000 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5})}{(1.000 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001210z^2)} \quad (3)$$

where  $z$  = depth beneath ground surface in meters. Eq. (3) yields essentially the same values for  $r_d$  as (2), but is easier to program and may be used in routine engineering practice.

I. M. Idriss [Transportation Research Board (TRB) (1999)] suggested a new procedure for determining magnitude-dependent values of  $r_d$ . Application of these  $r_d$  require use of a corresponding set of magnitude scaling factors that are compatible with the new  $r_d$ . Because these  $r_d$  were developed after the workshop and have not been independently evaluated by other experts, the workshop participants chose not to recommend the new factors at this time.

## EVALUATION OF LIQUEFACTION RESISTANCE (CRR)

A major focus of the workshop was on procedures for evaluating liquefaction resistance. A plausible method for evaluating CRR is to retrieve and test undisturbed soil specimens in the laboratory. Unfortunately, in situ stress states generally cannot be reestablished in the laboratory, and specimens of granular soils retrieved with typical drilling and sampling techniques are too disturbed to yield meaningful results. Only through specialized sampling techniques, such as ground freezing, can sufficiently undisturbed specimens be obtained. The cost of such procedures is generally prohibitive for all but the most critical projects. To avoid the difficulties associated with sampling and laboratory testing, field tests have become the state-of-practice for routine liquefaction investigations.

Several field tests have gained common usage for evaluation of liquefaction resistance, including the standard penetration test (SPT), the cone penetration test (CPT), shear-wave velocity measurements ( $V_s$ ), and the Becker penetration test (BPT). These tests were discussed at the workshop, along with associated criteria for evaluating liquefaction resistance. The participants made a conscientious attempt to correlate liquefaction resistance criteria from each of the various field tests to provide generally consistent results, no matter which test is applied. SPTs and CPTs are generally preferred because of the more extensive databases and past experience, but the other tests may be applied at sites underlain by gravelly sediment

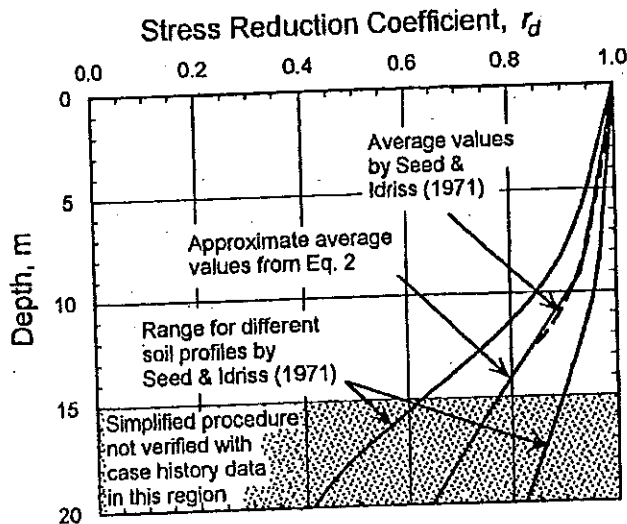


FIG. 1.  $r_d$  versus Depth Curves Developed by Seed and Idriss (1971) with Added Mean-Value Lines Plotted from Eq. (2)

TABLE 1. Comparison of Advantages and Disadvantages of Various Field Tests for Assessment of Liquefaction Resistance

Feature (1)	Test Type			
	SPT (2)	CPT (3)	$V_s$ (4)	BPT (5)
Past measurements at liquefaction sites	Abundant	Abundant	Limited	Sparse
Type of stress-strain behavior influencing test	Partially drained, large strain	Drained, large strain	Small strain	Partially drained, large strain
Quality control and repeatability	Poor to good	Very good	Good	Poor
Detection of variability of soil deposits	Good for closely spaced tests	Very good	Fair	Fair
Soil types in which test is recommended	Nongravel	Nongravel	All	Primarily gravel
Soil sample retrieved	Yes	No	No	No
Test measures index or engineering property	Index	Index	Engineering	Index

or where access by large equipment is limited. Primary advantages and disadvantages of each test are listed in Table 1.

## SPT

Criteria for evaluation of liquefaction resistance based on the SPT have been rather robust over the years. Those criteria are largely embodied in the CSR versus  $(N_1)_{60}$  plot reproduced in Fig. 2.  $(N_1)_{60}$  is the SPT blow count normalized to an overburden pressure of approximately 100 kPa (1 ton/sq ft) and a hammer energy ratio or hammer efficiency of 60%. The normalization factors for these corrections are discussed in the section entitled Other Corrections. Fig. 2 is a graph of calculated CSR and corresponding  $(N_1)_{60}$  data from sites where liquefaction effects were or were not observed following past earthquakes with magnitudes of approximately 7.5. CRR curves on this graph were conservatively positioned to separate regions with data indicative of liquefaction from regions with data indicative of nonliquefaction. Curves were developed for granular soils with the fines contents of 5% or less, 15%, and 35% as shown on the plot. The CRR curve for fines contents <5% is the basic penetration criterion for the simplified procedure and is referred to hereafter as the "SPT clean-sand base curve." The CRR curves in Fig. 2 are valid only for magnitude 7.5 earthquakes. Scaling factors to adjust CRR curves to other magnitudes are addressed in a later section of this report.

### SPT Clean-Sand Base Curve

Several changes to the SPT criteria are recommended by the workshop participants. The first change is to curve the trajectory of the clean-sand base curve at low  $(N_1)_{60}$  to a projected intercept of about 0.05 (Fig. 2). This adjustment reshapes the clean-sand base curve to achieve greater consistency with CRR curves developed for the CPT and shear-wave velocity procedures. Seed and Idriss (1982) projected the original curve through the origin, but there were few data to constrain the

curve in the lower part of the plot. A better fit to the present empirical data is to bow the lower end of the base curve as indicated in Fig. 2.

At the University of Texas, A. F. Rauch (personal communication, 1998), approximated the clean-sand base curve plotted in Fig. 2 by the following equation:

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \cdot (N_1)_{60} + 45]^2} - \frac{1}{200} \quad (4)$$

This equation is valid for  $(N_1)_{60} < 30$ . For  $(N_1)_{60} \geq 30$ , clean granular soils are too dense to liquefy and are classed as non-liquefiable. This equation may be used in spreadsheets and other analytical techniques to approximate the clean-sand base curve for routine engineering calculations.

### Influence of Fines Content

In the original development, Seed et al. (1985) noted an apparent increase of CRR with increased fines content. Whether this increase is caused by an increase of liquefaction resistance or a decrease of penetration resistance is not clear. Based on the empirical data available, Seed et al. developed CRR curves for various fines contents reproduced in Fig. 2. A revised correction for fines content was developed by workshop attendees to better fit the empirical database and to better support computations with spreadsheets and other electronic computational aids.

The workshop participants recommend (5) and (6) as approximate corrections for the influence of fines content (FC) on CRR. Other grain characteristics, such as soil plasticity, may affect liquefaction resistance as well as fines content, but widely accepted corrections for these factors have not been developed. Hence corrections based solely on fines content should be used with engineering judgment and caution. The following equations were developed by I. M. Idriss with the assistance of R. B. Seed for correction of  $(N_1)_{60}$  to an equivalent clean sand value,  $(N_1)_{60cs}$ :

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60} \quad (5)$$

where  $\alpha$  and  $\beta$  = coefficients determined from the following relationships:

$$\alpha = 0 \quad \text{for FC} \leq 5\% \quad (6a)$$

$$\alpha = \exp[1.76 - (190/FC^2)] \quad \text{for } 5\% < FC < 35\% \quad (6b)$$

$$\alpha = 5.0 \quad \text{for FC} \geq 35\% \quad (6c)$$

$$\beta = 1.0 \quad \text{for FC} \leq 5\% \quad (7a)$$

$$\beta = [0.99 + (FC^{1.5}/1,000)] \quad \text{for } 5\% < FC < 35\% \quad (7b)$$

$$\beta = 1.2 \quad \text{for FC} \geq 35\% \quad (7c)$$

These equations may be used for routine liquefaction resistance calculations. A back-calculated curve for a fines content of 35% is essentially congruent with the 35% curve plotted in Fig. 2. The back-calculated curve for a fines contents of 15% plots to the right of the original 15% curve.

### Other Corrections

Several factors in addition to fines content and grain characteristics influence SPT results, as noted in Table 2. Eq. (8) incorporates these corrections

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S \quad (8)$$

where  $N_m$  = measured standard penetration resistance;  $C_N$  = factor to normalize  $N_m$  to a common reference effective overburden stress;  $C_E$  = correction for hammer energy ratio (ER);  $C_B$  = correction factor for borehole diameter;  $C_R$  = correction

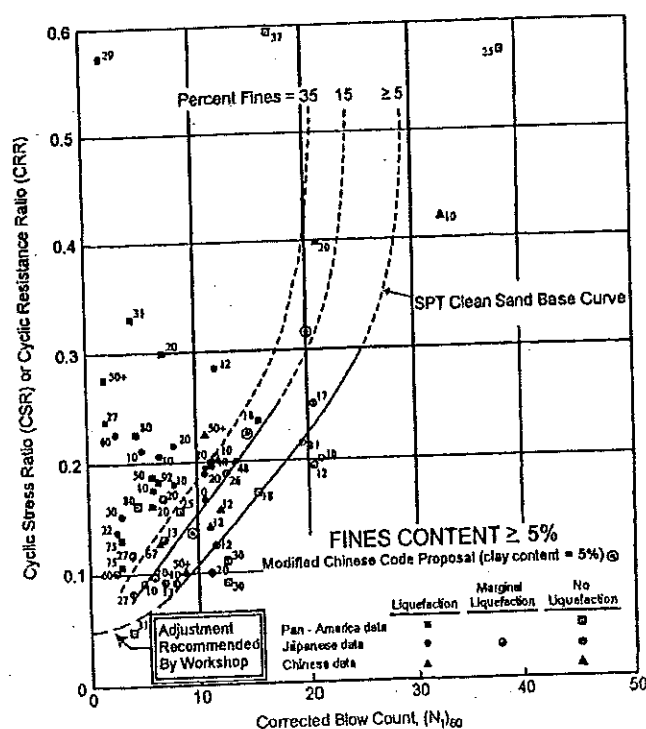


FIG. 2. SPT Clean-Sand Base Curve for Magnitude 7.5 Earthquakes with Data from Liquefaction Case Histories (Modified from Seed et al. 1985)

TABLE 2. Corrections to SPT (Modified from Skempton 1986) as Listed by Robertson and Wride (1998)

Factor (1)	Equipment variable (2)	Term (3)	Correction (4)
Overburden pressure	—	$C_N$	$(P_a/\sigma'_{vo})^{0.5}$
Overburden pressure	—	$C_N$	$C_N \leq 1.7$
Energy ratio	Donut hammer	$C_E$	0.5–1.0
Energy ratio	Safety hammer	$C_E$	0.7–1.2
Energy ratio	Automatic-trip Donut-type hammer	$C_E$	0.8–1.3
Borehole diameter	65–115 mm	$C_R$	1.0
Borehole diameter	150 mm	$C_R$	1.05
Borehole diameter	200 mm	$C_R$	1.15
Rod length	<3 m	$C_R$	0.75
Rod length	3–4 m	$C_R$	0.8
Rod length	4–6 m	$C_R$	0.85
Rod length	6–10 m	$C_R$	0.95
Rod length	10–30 m	$C_R$	1.0
Sampling method	Standard sampler	$C_S$	1.0
Sampling method	Sampler without liners	$C_S$	1.1–1.3

factor for rod length; and  $C_S$  = correction for samplers with or without liners.

Because SPT  $N$ -values increase with increasing effective overburden stress, an overburden stress correction factor is applied (Seed and Idriss 1982). This factor is commonly calculated from the following equation (Liao and Whitman 1986a):

$$C_N = (P_a/\sigma'_{vo})^{0.5} \quad (9)$$

where  $C_N$  normalizes  $N_m$  to an effective overburden pressure  $\sigma'_{vo}$  of approximately 100 kPa (1 atm)  $P_a$ .  $C_N$  should not exceed a value of 1.7 [A maximum value of 2.0 was published in the National Center for Earthquake Engineering Research (NCEER) workshop proceedings (Youd and Idriss 1997), but later was reduced to 1.7 by consensus of the workshop participants] Kayen et al. (1992) suggested the following equation, which limits the maximum  $C_N$  value to 1.7, and in these writers' opinion, provides a better fit to the original curve specified by Seed and Idriss (1982):

$$C_N = 2.2/(1.2 + \sigma'_{vo}/P_a) \quad (10)$$

Either equation may be used for routine engineering applications.

The effective overburden pressure  $\sigma'_{vo}$  applied in (9) and (10) should be the overburden pressure at the time of drilling and testing. Although a higher ground-water level might be used for conservatism in the liquefaction resistance calculations, the  $C_N$  factor must be based on the stresses present at the time of the testing.

The  $C_N$  correction factor was derived from SPT performed in test bins with large sand specimens subjected to various confining pressures (Gibbs and Holtz 1957; Marcuson and Bieganousky 1997a,b). The results of several of these tests are reproduced in Fig. 3 in the form of  $C_N$  curves versus effective overburden stress (Castro 1995). These curves indicate considerable scatter of results with no apparent correlation of  $C_N$  with soil type or gradation. The curves from looser sands, however, lie in the lower part of the  $C_N$  range and are reasonably approximated by (9) and (10) for low effective overburden pressures [200 kPa (<2 tsf)]. The workshop participants endorsed the use of (9) for calculation of  $C_N$ , but acknowledged that for overburden pressures >200 kPa (2 tsf) the results are uncertain. Eq. (10) provides a better fit for overburden pressures up to 300 kPa (3 tsf). For pressures >300 kPa (3 tsf), the uncertainty is so great that (9) should not be applied. At these high pressures, which are generally below the depth for which the simplified procedure has been verified,  $C_N$  should be estimated by other means.

Another important factor is the energy transferred from the

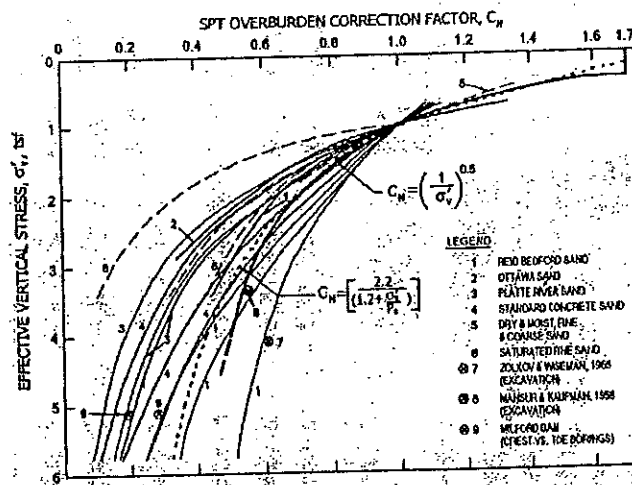


FIG. 3.  $C_N$  Curves for Various Sands Based on Field and Laboratory Test Data along with Suggested  $C_N$  Curve Determined from Eqs. (9) and (10) (Modified from Castro 1995)

falling hammer to the SPT sampler. An ER of 60% is generally accepted as the approximate average for U.S. testing practice and as a reference value for energy corrections. The ER delivered to the sampler depends on the type of hammer, anvil, lifting mechanism, and the method of hammer release. Approximate correction factors ( $C_E$  = ER/60) to modify the SPT results to a 60% energy ratio for various types of hammers and anvils are listed in Table 2. Because of variations in drilling and testing equipment and differences in testing procedures, a rather wide range in the energy correction factor  $C_E$  has been observed as noted in the table. Even when procedures are carefully monitored to conform to established standards, such as ASTM D 1586-99, some variation in  $C_E$  may occur because of minor variations in testing procedures. Measured energies at a single site indicate that variations in energy ratio between blows or between tests in a single borehole typically vary by as much as 10%. The workshop participants recommend measurement of the hammer energy frequently at each site where the SPT is used. Where measurements cannot be made, careful observation and notation of the equipment and procedures are required to estimate a  $C_E$  value for use in liquefaction resistance calculations. Use of good-quality testing equipment and carefully controlled testing procedures conforming to ASTM D 1586-99 will generally yield more consistent energy ratios and  $C_E$  with values from the upper parts of the ranges listed in Table 2.

Skempton (1986) suggested and Robertson and Wride (1998) updated correction factors for rod lengths <10 m, borehole diameters outside the recommended interval (65–125 mm), and sampling tubes without liners. Range for these correction factors are listed in Table 2. For liquefaction resistance calculations and rod lengths <3 m, a  $C_R$  of 0.75 should be applied as was done by Seed et al. (1985) in formulating the simplified procedure. Although application of rod-length correction factors listed in Table 2 will give more precise  $(N_1)_{60}$  values, these corrections may be neglected for liquefaction resistance calculations for rod lengths between 3 and 10 m because rod-length corrections were not applied to SPT test data from these depths in compiling the original liquefaction case history databases. Thus rod-length corrections are implicitly incorporated into the empirical SPT procedure.

A final change recommended by workshop participants is the use of revised magnitude scaling factors rather than the original Seed and Idriss (1982) factors to adjust CRR for earth-

quake magnitudes other than 7.5. Magnitude scaling factors are addressed later in this report.

## CPT

A primary advantage of the CPT is that a nearly continuous profile of penetration resistance is developed for stratigraphic interpretation. The CPT results are generally more consistent and repeatable than results from other penetration tests listed in Table 1. The continuous profile also allows a more detailed definition of soil layers than the other tools listed in the table. This stratigraphic capability makes the CPT particularly advantageous for developing liquefaction-resistance profiles. Interpretations based on the CPT, however, must be verified with a few well-placed boreholes preferably with standard penetration tests, to confirm soil types and further verify liquefaction-resistance interpretations.

Fig. 4 provides curves prepared by Robertson and Wride (1998) for direct determination of CRR for clean sands ( $FC \leq 5\%$ ) from CPT data. This figure was developed from CPT case history data compiled from several investigations, including those by Stark and Olson (1995) and Suzuki et al. (1995). The chart, valid for magnitude 7.5 earthquakes only, shows calculated cyclic resistance ratio plotted as a function of dimensionless, corrected, and normalized CPT resistance  $q_{c1N}$  from sites where surface effects of liquefaction were or were not observed following past earthquakes. The CRR curve conservatively separates regions of the plot with data indicative of liquefaction from regions indicative of nonliquefaction.

Based on a few misclassified case histories from the 1989 Loma Prieta earthquake, I. M. Idriss suggested that the clean sand curve in Fig. 4 should be shifted to the right by 10–15%. However, a majority of workshop participants supported a curve in its present position, for three reasons. First, purpose of the workshop was to recommend criteria that yield roughly equivalent CRR for the field tests listed in Table 1. Shifting the base curve to the right makes the CPT criteria generally more conservative. For example, for  $(N_1)_{60} > 5$ ,  $q_{c1N}/(N_1)_{60}$  ratios between the two clean-sand base curves, plotted in Figs. 4 and 2, respectively, range from 5 to 8—values that are slightly higher than those expected for clean sands. Shifting the CPT base curve to the right by 10 to 15% would increase those ratios to unusually high values ranging from 6 to 9.

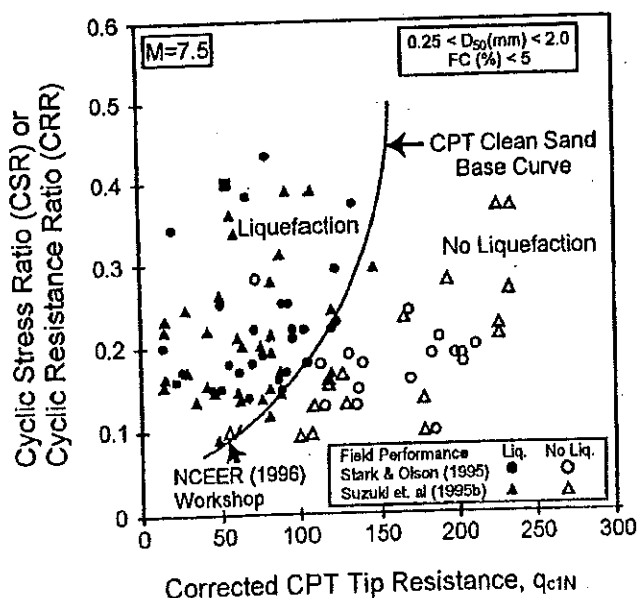


FIG. 4. Curve Recommended for Calculation of CRR from CPT Data along with Empirical Liquefaction Data from Compiled Case Histories (Reproduced from Robertson and Wride 1998)

Second, base curves, such as those plotted in Figs. 2 and 4, were intended to be conservative, but not necessarily to encompass every data point on the plot. Thus the presence of a few points beyond the base curve should be allowable. Finally, several studies have confirmed that the CPT criteria in Fig. 4 are generally conservative. Robertson and Wride (1998) verified these criteria against SPT and other data from sites they investigated. Gilstrap and Youd (1998) compared calculated liquefaction resistances against field performance at 19 sites and concluded that the CPT criteria correctly predicted the occurrence or nonoccurrence of liquefaction with >85% reliability.

The clean-sand base curve in Fig. 4 may be approximated by the following equation (Robertson and Wride 1998):

$$\text{If } (q_{c1N})_{cr} < 50 \quad CRR_{7.5} = 0.833[(q_{c1N})_{cr}/1,000] + 0.05 \quad (11a)$$

$$\text{If } 50 \leq (q_{c1N})_{cr} < 160 \quad CRR_{7.5} = 93[(q_{c1N})_{cr}/1,000]^3 + 0.08 \quad (11b)$$

where  $(q_{c1N})_{cr}$  = clean-sand cone penetration resistance normalized to approximately 100 kPa (1 atm).

### Normalization of Cone Penetration Resistance

The CPT procedure requires normalization of tip resistance using (12) and (13). This transformation yields normalized, dimensionless cone penetration resistance  $q_{c1N}$

$$q_{c1N} = C_Q(q_c/P_a) \quad (12)$$

where

$$C_Q = (P_a/\sigma'_{vo})^n \quad (13)$$

and where  $C_Q$  = normalizing factor for cone penetration resistance;  $P_a$  = 1 atm of pressure in the same units used for  $\sigma'_{vo}$ ;  $n$  = exponent that varies with soil type; and  $q_c$  = field cone penetration resistance measured at the tip. At shallow depths  $C_Q$  becomes large because of low overburden pressure; however, values >1.7 should not be applied. As noted in the following paragraphs, the value of the exponent  $n$  varies from 0.5 to 1.0, depending on the grain characteristics of the soil (Olsen 1997).

The CPT friction ratio (sleeve resistance  $f$ , divided by cone tip resistance  $q_c$ ) generally increases with increasing fines content and soil plasticity, allowing rough estimates of soil type and fines content to be determined from CPT data. Robertson and Wride (1998) constructed the chart reproduced in Fig. 5 for estimation of soil type. The boundaries between soil types 2–7 can be approximated by concentric circles and can be used to account for effects of soil characteristics on  $q_{c1N}$  and CRR. The radius of these circles, termed the soil behavior type index  $I_c$  is calculated from the following equation:

$$I_c = [(3.47 - \log Q)^2 + (1.22 + \log F)^2]^{0.5} \quad (14)$$

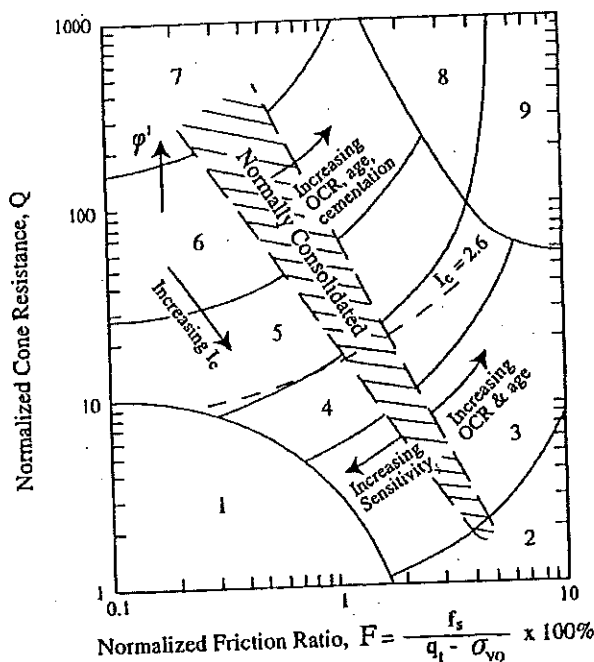
where

$$Q = [(q_c - \sigma_{vo})/P_a][(P_a/\sigma'_{vo})^n] \quad (15)$$

and

$$F = [f/(q_c - \sigma_{vo})] \times 100\% \quad (16)$$

The soil behavior chart in Fig. 5 was developed using an exponent  $n$  of 1.0, which is the appropriate value for clayey soil types. For clean sands, however, an exponent value of 0.5 is more appropriate, and a value intermediate between 0.5 and 1.0 would be appropriate for silts and sandy silts. Robertson and Wride recommended the following procedure for calculating the soil behavior type index  $I_c$ . The first step is to differentiate soil types characterized as clays from soil types characterized as sands and silts. This differentiation is performed by assuming an exponent  $n$  of 1.0 (characteristic of clays) and calculating the dimensionless CPT tip resistance  $Q$  from the following equation:



1. Sensitive, fine grained
  2. Organic soils - peats
  3. Clays - silty clay to clay
  4. Silt mixtures - clayey silt to silty clay
  5. Sand mixtures - silty sand to sandy silt
  6. Sands - clean sand to silty sand
  7. Gravelly sand to dense sand
  8. Very stiff sand to clayey sand\*
  9. Very stiff, fine grained\*
- \*Heavily overconsolidated or cemented

FIG. 5. CPT-Based Soil Behavior-Type Chart Proposed by Robertson (1990)

$$Q = [(q_c - \sigma_{vo})/P_a][P_a/\sigma'_{vo}]^{1.0} = [(q_c - \sigma_{vo})/\sigma'_{vo}] \quad (17)$$

If the  $I_c$  calculated with an exponent of 1.0 is  $>2.6$ , the soil is classified as clayey and is considered too clay-rich to liquefy, and the analysis is complete. However, soil samples should be retrieved and tested to confirm the soil type and liquefaction resistance. Criteria such as the Chinese criteria might be applied to confirm that the soil is nonliquefiable. The so-called Chinese criteria, as defined by Seed and Idriss (1982), specify that liquefaction can only occur if all three of the following conditions are met:

1. The clay content (particles smaller than  $5 \mu$ ) is  $<15\%$  by weight.
2. The liquid limit is  $<35\%$ .
3. The natural moisture content is  $>0.9$  times the liquid limit.

If the calculated  $I_c$  is  $<2.6$ , the soil is most likely granular in nature, and therefore  $C_q$  and  $Q$  should be recalculated using an exponent  $n$  of 0.5.  $I_c$  should then be recalculated using (14). If the recalculated  $I_c$  is  $<2.6$ , the soil is classed as nonplastic and granular. This  $I_c$  is used to estimate liquefaction resistance, as noted in the next section. However, if the recalculated  $I_c$  is  $>2.6$ , the soil is likely to be very silty and possibly plastic. In this instance,  $q_{c1N}$  should be recalculated from (12) using an intermediate exponent  $n$  of 0.7 in (13).  $I_c$  is then recalculated from (14) using the recalculated value for  $q_{c1N}$ . This intermediate  $I_c$  is then used to calculate liquefaction resistance. In this instance, a soil sample should be retrieved and tested to verify the soil type and whether the soil is liquefiable by other criteria, such as the Chinese criteria.

Because the relationship between  $I_c$  and soil type is approx-

imate, the consensus of the workshop participants is that all soils with an  $I_c$  of 2.4 or greater should be sampled and tested to confirm the soil type and to test the liquefiability with other criteria. Also, soil layers characterized by an  $I_c > 2.6$ , but with a normalized friction ratio  $F < 1.0\%$  (region 1 of Fig. 5) may be very sensitive and should be sampled and tested. Although not technically liquefiable according to the Chinese criteria, such sensitive soils may suffer softening and strength loss during earthquake shaking.

#### Calculation of Clean-Sand Equivalent Normalized Cone Penetration Resistance ( $q_{c1N}$ )

The normalized penetration resistance ( $q_{c1N}$ ) for silty sands is corrected to an equivalent clean sand value ( $q_{c1N}$ ) by the following relationship:

$$(q_{c1N})_{cs} = K_c q_{c1N} \quad (18)$$

where  $K_c$ , the correction factor for grain characteristics, is defined by the following equation (Robertson and Wride 1998):

$$\text{for } I_c \leq 1.64 \quad K_c = 1.0 \quad (19a)$$

$$\text{for } I_c > 1.64 \quad K_c = -0.403I_c^4 + 5.581I_c^3 - 21.63I_c^2 + 33.75I_c - 17.88 \quad (19b)$$

The  $K_c$  curve defined by (19) is plotted in Fig. 6. For  $I_c > 2.6$ , the curve is shown as a dashed line, indicating that soils in this range of  $I_c$  are most likely too clay-rich or plastic to liquefy.

With an appropriate  $I_c$  and  $K_c$ , (11) and (19) can be used to calculate  $CRR_{7.5}$ . To adjust CRR to magnitudes other than 7.5, the calculated  $CRR_{7.5}$  is multiplied by an appropriate magnitude scaling factor. The same magnitude scaling factors are used with CPT data as with SPT data. Magnitude scaling factors are discussed in a later section of this report.

#### Olsen (1997) and Suzuki et al. (1995) Procedures

Olsen (1997), who pioneered many of the techniques for assessing liquefaction resistance from CPT soundings, suggested a somewhat different procedure for calculating CRR from CPT data. Reasons for recommending the Robertson and Wride (1998) procedure over that of Olsen are the ease of application and the ease with which relationships can be quantified for computer-aided calculations. Results from Olsen's procedure, however, are consistent with results from the procedure proposed here for shallow ( $<15$  m deep) sediment be-

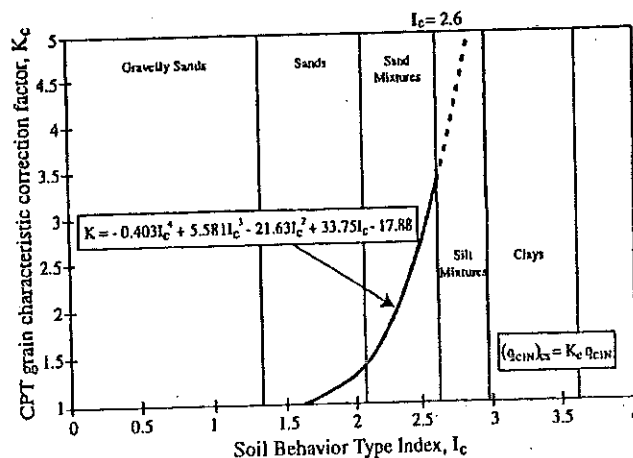


FIG. 6. Grain-Characteristic Correction Factor  $K_c$  for Determination of Clean-Sand Equivalent CPT Resistance (Reproduced from Robertson and Wride 1998)

neath level to gently sloping terrain. Olsen (1997) noted that almost any CPT normalization technique will give results consistent with his normalization procedure for soil layers in the 3–15 m depth range. For deeper layers, significant differences may develop between the two procedures. Those depths are also beyond the depth for which the simplified procedure has been verified. Hence any procedure based on the simplified procedure yields rather uncertain results at depths >15 m.

Suzuki et al. (1995) also developed criteria for evaluating CRR from CPT data. Those criteria are slightly more conservative than those of Robertson and Wride (1998) and were considered by the latter investigators in developing the criteria recommended herein.

#### Correction of Cone Penetration Resistance for Thin Soil Layers

Theoretical as well as laboratory studies indicate that CPT tip resistance is influenced by softer soil layers above or below the cone tip. As a result, measured CPT tip resistance is smaller in thin layers of granular soils sandwiched between softer layers than in thicker layers of the same granular soil. The amount of the reduction of penetration resistance in soft layers is a function of the thickness of the softer layer and the stiffness of the stiffer layers.

Using a simplified elastic solution, Vreugdenhil et al. (1994) developed a procedure for estimating the thick-layer equivalent cone penetration resistance of thin stiff layers lying within softer strata. The correction applies only to thin stiff layers embedded within thick soft layers. Because the corrections have a reasonable trend, but appear rather large, Robertson and Fear (1995) recommended conservative corrections from the  $q_{cA}/q_{cB} = 2$  curve sketched in Fig. 7.

Further analysis of field data by Gonzalo Castro and Peter Robertson for the NCEER workshop indicates that corrections based on the  $q_{cA}/q_{cB} = 2$  curve may still be too large and not adequately conservative. They suggested, and the workshop participants agreed, that the lower bound of the range of field data plotted by G. Castro in Fig. 7 provides more conservative  $K_H$  values that should be used until further field studies and analyses indicate that higher values are viable. The equation for the lower bound of the field curve is

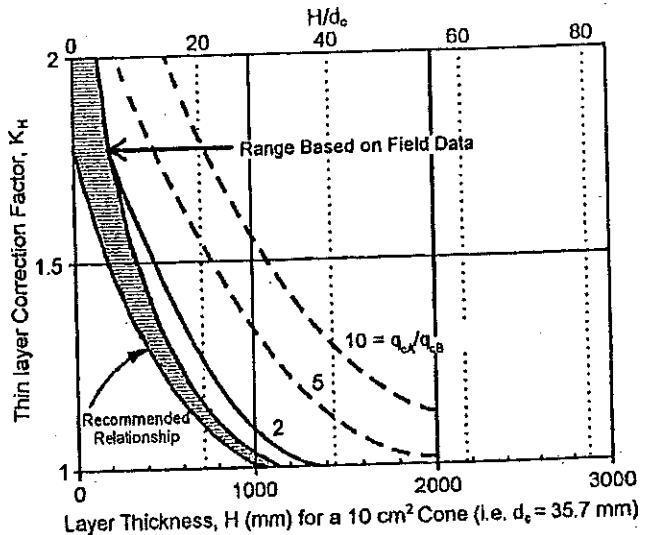
$$K_H = 0.25[(H/d_c)/17 - 1.77]^2 + 1.0 \quad (20)$$

where  $H$  = thickness of the interbedded layer in mm;  $q_{cA}$  and  $q_{cB}$  = cone resistances of the stiff and soft layers, respectively; and  $d_c$  = diameter of the cone in mm (Fig. 7).

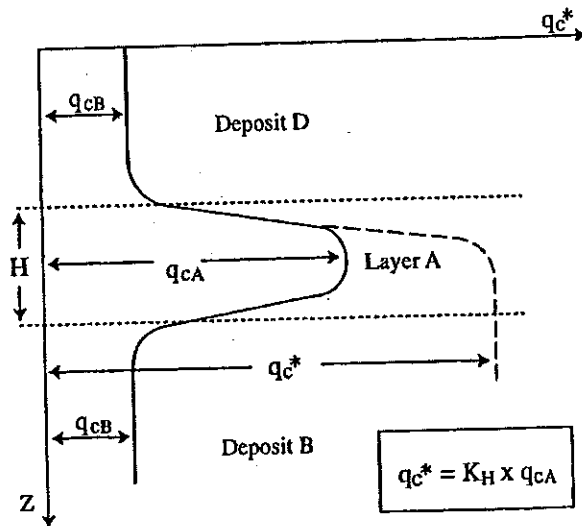
#### $V_s$

Andrus and Stokoe (1997, 2000) developed liquefaction resistance criteria from field measurements of shear wave velocity  $V_s$ . The use of  $V_s$  as a field index of liquefaction resistance is soundly based because both  $V_s$  and CRR are similarly, but not proportionally, influenced by void ratio, effective confining stresses, stress history, and geologic age. The advantages of using  $V_s$  include the following: (1)  $V_s$  measurements are possible in soils that are difficult to penetrate with CPT and SPT or to extract undisturbed samples, such as gravelly soils, and at sites where borings or soundings may not be permitted; (2)  $V_s$  is a basic mechanical property of soil materials, directly related to small-strain shear modulus; and (3) the small-strain shear modulus is a parameter required in analytical procedures for estimating dynamic soil response and soil-structure interaction analyses.

Three concerns arise when using  $V_s$  for liquefaction-resistance evaluations: (1) seismic wave velocity measurements are made at small strains, whereas pore-water pressure buildup and the onset of liquefaction are medium- to high-strain phenomena;



(a)



(b)

FIG. 7. Thin-Layer Correction Factor  $K_H$  for Determination of Equivalent Thick-Layer CPT Resistance (Modified from Robertson and Fear 1995)

(2) seismic testing does not provide samples for classification of soils and identification of nonliquefiable soft clay-rich soils; and (3) thin, low  $V_s$  strata may not be detected if the measurement interval is too large. Therefore the preferred practice is to drill sufficient boreholes and conduct in situ tests to detect and delineate thin liquefiable strata, nonliquefiable clay-rich soils, and silty soils above the ground-water table that might become liquefiable should the water table rise. Other tests, such as the SPT or CPT, are needed to detect liquefiable weakly cemented soils that may have high  $V_s$  values.

#### $V_s$ Criteria for Evaluating Liquefaction Resistance

Following the traditional procedures for correcting penetration resistance to account for overburden stress,  $V_s$  is also corrected to a reference overburden stress using the following equation (Sykora 1987; Kayen et al. 1992; Robertson et al. 1992):

$$V_{s1} = V_s \left( \frac{P_a}{\sigma'_{v0}} \right)^{0.25} \quad (21)$$

where  $V_{s1}$  = overburden-stress corrected shear wave velocity;  $P_a$  = atmospheric pressure approximated by 100 kPa (1 TSF);



and  $\sigma'_{vo}$  = initial effective vertical stress in the same units as  $P_a$ . Eq. (21) implicitly assumes a constant coefficient of earth pressure  $K'_0$  which is approximately 0.5 for sites susceptible to liquefaction. Application of (21) also implicitly assumes that  $V_s$  is measured with both the directions of particle motion and wave propagation polarized along principal stress directions and that one of those directions is vertical (Stokoe et al. 1985).

Fig. 8 compares seven CRR- $V_{s1}$  curves. The "best fit" curve by Tokimatsu and Uchida (1990) was determined from laboratory cyclic triaxial test results for various sands with <10% fines and 15 cycles of loading. The more conservative "lower bound" curve for Tokimatsu and Uchida's laboratory test results is also shown as a lower bound for liquefaction occurrences. The bounding curve by Robertson et al. (1992) was developed using field performance data from sites in Imperial Valley, Calif., along with data from four other sites. The curves by Kayen et al. (1992) and Lodge (1994) are from sites that did and did not liquefy during the 1989 Loma Prieta earthquake. Andrus and Stokoe's (1997) curve was developed for uncemented, Holocene-age soils with 5% or less fines using field performance data from 20 earthquakes and over 50 measurement sites. Andrus and Stokoe (2000) revised this curve based on new information and an expanded database that includes 26 earthquakes and more than 70 measurement sites.

Andrus and Stokoe (1997) proposed the following relationship between CRR and  $V_{s1}$ :

$$CRR = a \left( \frac{V_{s1}}{100} \right)^2 + b \left( \frac{1}{V_{s1}^* - V_{s1}} - \frac{1}{V_{s1}^*} \right) \quad (22)$$

where  $V_{s1}^*$  = limiting upper value of  $V_{s1}$  for liquefaction occurrence; and  $a$  and  $b$  are curve fitting parameters. The first parenthetical term of (22) is based on a modified relationship between  $V_{s1}$  and CSR for constant average cyclic shear strain suggested by R. Dobry (personal communication to R. D. Andrus, 1996). The second parenthetical term is a hyperbola with a small value at low  $V_{s1}$ , and a very large value as  $V_{s1}$  approaches  $V_{s1}^*$ , a constant limiting velocity for liquefaction of soils.

CRR versus  $V_{s1}$  curves recommended for engineering practice by Andrus and Stokoe (2000) for magnitude 7.5 earth-

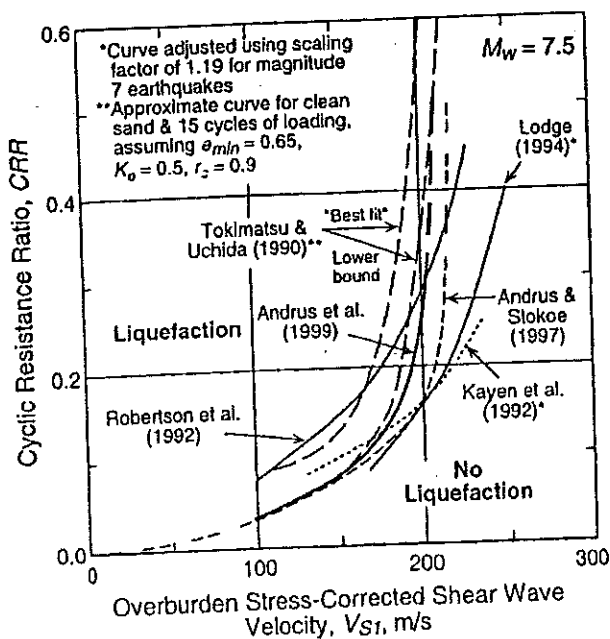


FIG. 8. Comparison of Seven Relationships between Liquefaction Resistance and Overburden Stress-Corrected Shear Wave Velocity for Granular Soils

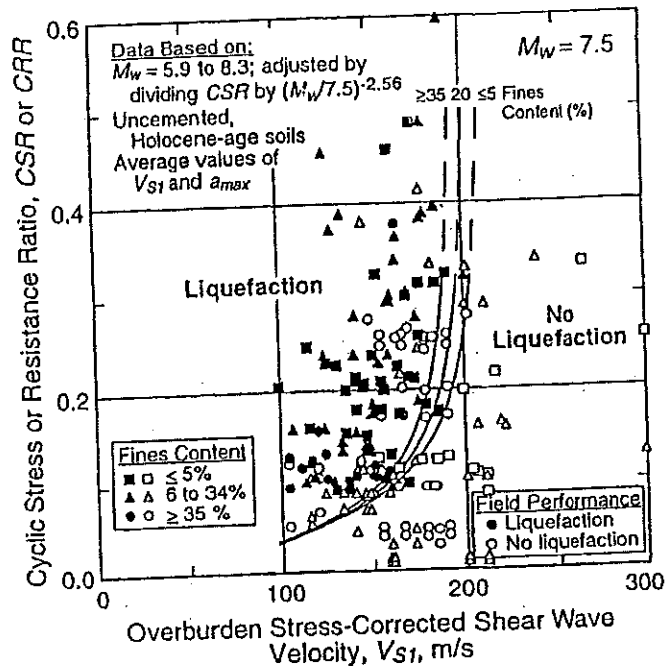


FIG. 9. Liquefaction Relationship Recommended for Clean, Uncemented Soils with Liquefaction Data from Compiled Case Histories (Reproduced from Andrus and Stokoe 2000)

quakes and uncemented Holocene-age soils with various fines contents are reproduced in Fig. 9. Also plotted and presented in Fig. 9 are points calculated from liquefaction case history information for magnitude 5.9–8.3 earthquakes. The three curves shown were determined through an iterative process of varying the values of  $a$  and  $b$  until nearly all the points indicative of liquefaction were bounded by the curves with the least number of nonliquefaction points plotted in the liquefaction region. The final values of  $a$  and  $b$  used to draw the curves were 0.022 and 2.8, respectively. Values of  $V_{s1}^*$  were assumed to vary linearly from 200 m/s for soils with fines content of 35% to 215 m/s for soils with fines content of 5% or less.

The recommended curves shown in Fig. 9 are dashed above CRR of 0.35 to indicate that field-performance data are limited in that range. Also, they do not extend much below 100 m/s, because there are no field data to support extending them to the origin. The calculated CRR is 0.033 for a  $V_{s1}$  of 100 m/s. This minimal CRR value is generally consistent with intercept CRR values assumed for the CPT and SPT procedures. Eq. (22) can be scaled to other magnitude values through use of magnitude scaling factors. These factors are discussed in a later section of this paper.

## BPT

Liquefaction resistance of nongravelly soils has been evaluated primarily through CPT and SPT, with occasional  $V_s$  measurements. CPT and SPT measurements, however, are not generally reliable in gravelly soils. Large gravel particles may interfere with the normal deformation of soil materials around the penetrometer and misleadingly increase penetration resistance. Several investigators have employed large-diameter penetrometers to surmount these difficulties; the Becker penetration test (BPT) in particular has become one of the more effectively and widely used larger tools. The BPT was developed in Canada in the late 1950s and consists of a 168-mm diameter, 3-m-long double-walled casing driven into the ground with a double-acting diesel-driven pile hammer. The hammer impacts are applied at the top of the casing and penetration is continuous. The Becker penetration resistance is



defined as the number of blows required to drive the casing through an increment of 300 mm.

The BPT has not been standardized, and several different types of equipment and procedures have been used. There are currently very few liquefaction sites from which BPT data have been obtained. Thus the BPT cannot be directly correlated with field behavior, but rather through estimating equivalent SPT  $N$ -values from BPT data and then applying evaluation procedures based on the SPT. This indirect method introduces substantial additional uncertainty into the calculated CRR.

To provide uniformity, Harder and Seed (1986) recommended newer AP-1000 drill rigs equipped with supercharged diesel hammers, 168-mm outside diameter casing, and a plugged bit. From several sites where both BPT and SPT tests were conducted in parallel soundings, Harder and Seed (1986) developed a preliminary correlation between Becker and standard penetration resistance [Fig. 10(a)]. Additional comparative data compiled since 1986 are plotted in Fig. 10(b). The original Harder and Seed correlation curve (solid line) is drawn in Fig. 10(b) along with dashed curves representing 20% over- and underpredictions of SPT blow counts. These plots indicate that SPT blow counts can be roughly estimated from BPT measurements. These plots indicate that although SPT blow counts can be roughly estimated from BPT measurements, there can be considerable uncertainty for calculating liquefaction resistance because the data scatter is greatest in the range of greatest importance [ $N$ -values of 0–30 blows/300 mm (ft)].

A major source of variation in BPT blow counts is deviation

in hammer energy. Rather than measuring hammer energy directly, Harder and Seed (1986) monitored bounce-chamber pressures and found that uniform combustion conditions (e.g., full throttle with a supercharger) correlated rather well with variations in Becker blow count. From this information, Harder and Seed developed an energy correction procedure based on measured bounce-chamber pressure.

Direct measurement of transmitted hammer energy could provide a more theoretically rigorous correction for Becker hammer efficiency. Sy and Campanella (1994) and Sy et al. (1995) instrumented a small length of Becker casing with strain gauges and accelerometers to measure transferred energy. They analyzed the recorded data with a pile-driving analyzer to determine strain, force, acceleration, and velocity. The transferred energy was determined by time integration of force times velocity. They were able to verify many of the variations in hammer energy previously identified by Harder and Seed (1986), including effects of variable throttle settings and energy transmission efficiencies of various drill rigs. However, they were unable to reduce the amount of scatter and uncertainty in converting BPT blow counts to SPT blow counts. Because the Sy and Campanella procedure requires considerably more effort than monitoring of bounce-chamber pressure without producing greatly improved results, the workshop participants agreed that the bounce-chamber technique is adequate for routine practice.

Friction along the driven casing also influences penetration resistance. Harder and Seed (1986) did not directly evaluate the effect of casing friction; hence, the correlation in Fig. 10(b) intrinsically incorporates an unknown amount of casing friction. However, casing friction remains a concern for depths  $>30$  m and for measurement of penetration resistance in soft soils underlying thick deposits of dense soil. Either of these circumstances could lead to greater casing friction than is intrinsically incorporated in the Harder and Seed correlation.

The following procedures are recommended for routine practice: (1) the BPT should be conducted with newer AP-1000 drill rigs equipped with supercharged diesel hammers to drive plugged 168-mm outside diameter casing; (2) bounce-chamber pressures should be monitored and adjustments made to measured BPT blow counts to account for variations in diesel hammer combustion efficiency—for most routine applications, correlations developed by Harder and Seed (1986) may be used for these adjustments; and (3) the influence of some casing friction is indirectly accounted for in the Harder and Seed BPT-SPT correlation. This correlation, however, has not been verified and should not be used for depths  $>30$  m or for sites with thick dense deposits overlying loose sands or gravels. For these conditions, mudded boreholes may be needed to reduce casing friction, or specially developed local correlations or sophisticated wave-equation analyses may be applied to quantify frictional effects.

#### MAGNITUDE SCALING FACTORS (MSFs)

The clean-sand base or CRR curves in Figs. 2 (SPT), 4 (CPT), and 10 ( $V_{s1}$ ) apply only to magnitude 7.5 earthquakes. To adjust the clean-sand curves to magnitudes smaller or larger than 7.5, Seed and Idriss (1982) introduced correction factors termed "magnitude scaling factors (MSFs)." These factors are used to scale the CRR base curves upward or downward on CRR versus ( $N_1$ )<sub>60</sub>,  $q_{c1N_0}$ , or  $V_{s1}$  plots. Conversely, magnitude weighting factors, which are the inverse of magnitude scaling factors, may be applied to correct CSR for magnitude. Either correcting CRR via magnitude scaling factors, or correcting CSR via magnitude weighting factors, leads to the same final result. Because the original papers by Seed and Idriss were written in terms of magnitude scaling factors, the use of magnitude scaling factors is continued in this report.

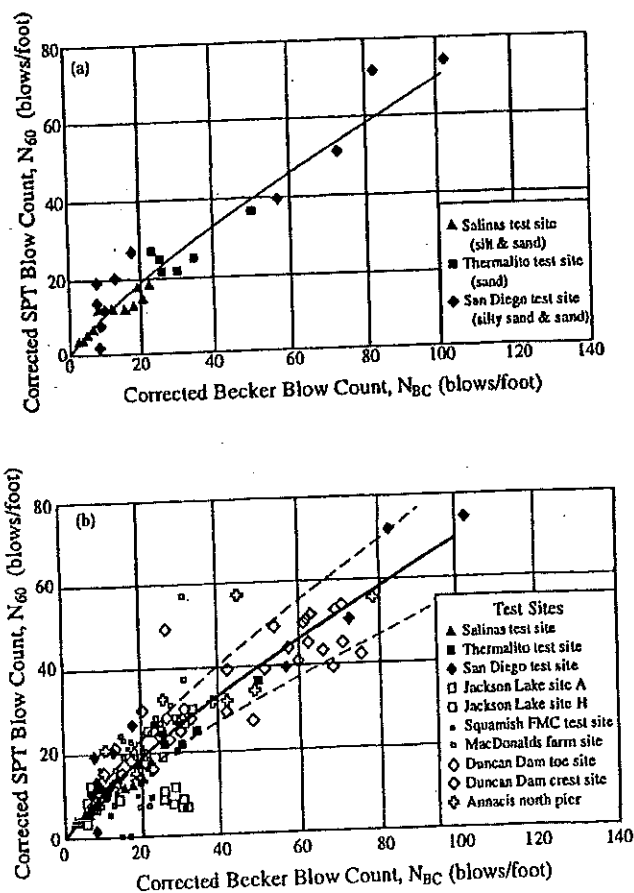


FIG. 10. Correlation between Corrected Becker Penetration Resistance  $N_{Bc}$  and Corrected SPT Resistance  $N_{60}$ : (a) Harder and Seed (1986); (b) Data from Additional Sites (Reproduced from Harder 1997)

To illustrate the influence of magnitude scaling factors on calculated hazard, the equation for factor of safety (FS) against liquefaction is written in terms of CRR, CSR, and MSF as follows:

$$FS = (CRR_{7.5}/CSR)MSF \quad (23)$$

where CSR = calculated cyclic stress ratio generated by the earthquake shaking; and  $CRR_{7.5}$  = cyclic resistance ratio for magnitude 7.5 earthquakes.  $CRR_{7.5}$  is determined from Fig. 2 or (4) for SPT data, Fig. 4 or (11) for CPT data, or Fig. 9 or (22) for  $V_{si}$  data.

### Seed and Idriss (1982) Scaling Factors

Because of the limited amount of field liquefaction data available in the 1970s, Seed and Idriss (1982) were unable to adequately constrain bounds between liquefaction and non-liquefaction regions on CRR plots for magnitudes other than 7.5. Consequently, they developed a set of MSF from average numbers of loading cycles for various earthquake magnitudes and laboratory test results. A representative curve developed by these investigators, showing the number of loading cycles required to generate liquefaction for a given CSR, is reproduced in Fig. 11. The average number of loading cycles for various magnitudes of earthquakes are also noted on the plot. The initial set of magnitude scaling factors was derived by dividing CSR values on the representative curve for the number of loading cycles corresponding to a given earthquake magnitude by the CSR for 15 loading cycles (equivalent to a magnitude 7.5 earthquake). These scaling factors are listed in column 2 of Table 3 and are plotted in Fig. 12. These MSFs have been routinely applied in engineering practice since their introduction in 1982.

### Revised Idriss Scaling Factors

In preparing his H. B. Seed Memorial Lecture, I. M. Idriss reevaluated the data that he and the late Professor Seed used

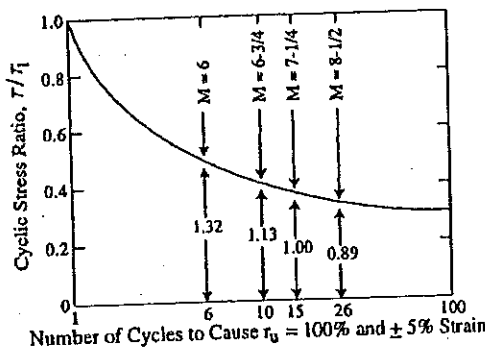


FIG. 11. Representative Relationship between CSR and Number of Cycles to Cause Liquefaction (Reproduced from Seed and Idriss 1982)

to calculate the original (1982) magnitude scaling factors. In so doing, Idriss replotted the data on a log-log plot and suggested that the data should plot as a straight line. He noted, however, that one outlying point had strongly influenced the original analysis, causing the original plot to be nonlinear and characterized by unduly low MSF values for magnitudes <7.5. Based on this reevaluation, Idriss defined a revised set of magnitude scaling factors listed in column 3 of Table 3 and plotted in Fig. 12. The revised MSFs are defined by the following equation:

$$MSF = 10^{2.24/M_w^{2.56}} \quad (24)$$

The workshop participants recommend these revised scaling factors as a lower bound for MSF values.

The revised scaling factors are significantly higher than the original scaling factors for magnitudes <7.5 and somewhat lower than the original factors for magnitudes >7.5. Relative to the original scaling factors, the revised factors lead to a reduced calculated liquefaction hazard for magnitudes <7.5, but increase calculated hazard for magnitudes >7.5.

### Ambroseys (1988) Scaling Factors

Field performance data collected since the 1970s for magnitudes <7.5 indicate that the original Seed and Idriss (1982) scaling factors are overly conservative. For example, Ambroseys (1988) analyzed liquefaction data compiled through the mid-1980s and plotted calculated cyclic stress ratios for sites that did or did not liquefy versus  $(N_1)_{60}$ . From these plots, Ambroseys developed empirical exponential equations that define CRR as a function of  $(N_1)_{60}$  and moment magnitude  $M_w$ . By holding the value of  $(N_1)_{60}$  constant in the equations and taking the ratio of CRR determined for various magnitudes of earthquakes to the CRR for magnitude 7.5 earthquakes, Am-

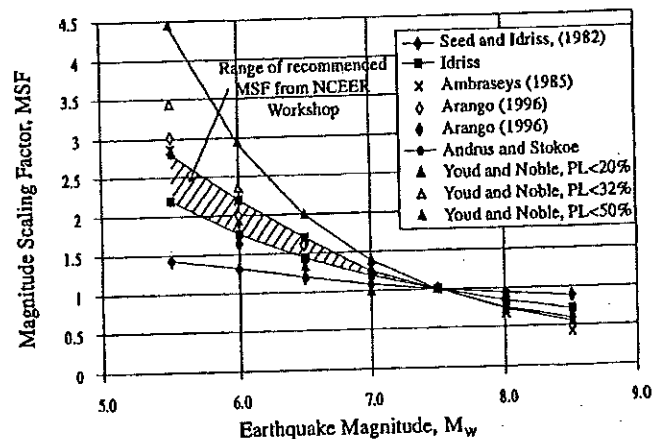


FIG. 12. Magnitude Scaling Factors Derived by Various Investigators (Reproduced from Youd and Noble 1997a)

TABLE 3. Magnitude Scaling Factor Values Defined by Various Investigators (Youd and Noble 1997a)

Magnitude, $M$ (1)	Seed and Idriss (1982) (2)	Idriss <sup>a</sup> (3)	Ambroseys (1988) (4)	Arango (1996)		Andrus and Stokoe (1997) (7)	Youd and Noble (1997b)		
				Distance based (5)	Energy based (6)		$P_L < 20\%$ (8)	$P_L < 32\%$ (9)	$P_L < 50\%$ (10)
5.5	1.43	2.20	2.86	3.00	2.20	2.8	2.86	3.42	4.44
6.0	1.32	1.76	2.20	2.00	1.65	2.1	1.93	2.35	2.92
6.5	1.19	1.44	1.69	1.60	1.40	1.6	1.34	1.66	1.99
7.0	1.08	1.19	1.30	1.25	1.10	1.25	1.00	1.20	1.39
7.5	1.00	1.00	1.00	1.00	1.00	1.00	—	—	1.00
8.0	0.94	0.84	0.67	0.75	0.85	0.87	—	—	0.73?
8.5	0.89	0.72	0.44	—	—	0.65?	—	—	0.56?

Note: ? = Very uncertain values.

<sup>a</sup>1995 Seed Memorial Lecture, University of California at Berkeley (I. M. Idriss, personal communication to T. L. Youd, 1997).

braseys derived the magnitude scaling factors listed in column 4 of Table 3 and plotted in Fig. 12. For magnitudes <7.5, the MSFs suggested by Ambraseys are significantly larger than both the original factors developed by Seed and Idriss (column 2, Table 3) and the revised factors suggested by Idriss (column 3). Because they are based on observational data, these factors have validity for estimating liquefaction hazard; however, they have not been widely used in engineering practice.

For magnitudes >7.5, Ambraseys factors are significantly lower and much more conservative than the original (Seed and Idriss 1982) and Idriss's revised scaling factors. Because there are few data to constrain Ambraseys' scaling factors for magnitudes >7.5, they are not recommended for hazard evaluation for large earthquakes.

### Arango (1996) Scaling Factors

Arango (1996) developed two sets of magnitude scaling factors. The first set (column 5, Table 3) is based on furthest observed liquefaction effects from the seismic energy source, the estimated average peak accelerations at those distant sites, and the seismic energy required to cause liquefaction. The second set (column 6, Table 3) was developed from energy concepts and the relationship derived by Seed and Idriss (1982) between numbers of significant stress cycles and earthquake magnitude. The MSFs listed in column 5 are similar in value (within about 10%) to the MSFs of Ambraseys (column 4), and the MSFs listed in column 6 are similar in value (within about 10%) to the revised MSFs proposed by Idriss (column 3).

### Andrus and Stokoe (1997) Scaling Factors

From their studies of liquefaction resistance as a function of shear wave velocity  $V_s$ , Andrus and Stokoe (1997) drew bounding curves and developed (22) for calculating CRR from  $V_s$  for magnitude 7.5 earthquakes. These investigators drew similar bounding curves for sites where surface effects of liquefaction were or were not observed for earthquakes with magnitudes of 6, 6.5, and 7. The positions of the CRR curves were visually adjusted on each graph until a best-fit bound was obtained. Magnitude scaling factors were then estimated by taking the ratio of CRR for a given magnitude to the CRR for magnitude 7.5 earthquakes. These MSFs are quantified by the following equation:

$$MSF = (M_w/7.5)^{-2.56} \quad (25)$$

MSFs for magnitudes <6 and >7.5 were extrapolated from this equation. The derived MSFs are listed in column 7 of Table 3, and plotted in Fig. 12. For magnitudes <7.5, the MSFs proposed by Andrus and Stokoe are rather close in value (within about 5%) to the MSFs proposed by Ambraseys. For magnitudes >7.5, the Andrus and Stokoe MSFs are slightly smaller than the revised MSFs proposed by Idriss.

### Youd and Noble (1997a) Scaling Factors

Youd and Noble (1997a) used a probabilistic or logistic analysis to analyze case history data from sites where effects of liquefaction were or were not reported following past earthquakes. This analysis yielded the following equation, which was updated after publication of the NCEER proceedings (Youd and Idriss 1997):

$$\text{Logit}(P_L) = \ln(P_L/(1 - P_L)) = -7.0351 + 2.1738M_w - 0.2678(N_1)_{60\text{cr}} + 3.0265 \ln \text{CRR} \quad (26)$$

where  $P_L$  = probability that liquefaction occurred;  $1 - P_L$  = probability that liquefaction did not occur; and  $(N_1)_{60\text{cr}}$  = cor-

rected equivalent clean-sand blow count. For magnitudes <7.5, Youd and Noble recommended direct application of this equation to calculate the CRR for a given probability of liquefaction. In lieu of direct application, Youd and Noble defined three sets of MSFs for use with the simplified procedure. These MSFs are for probabilities of liquefaction occurrence <20, 32, and 50%, respectively, and are defined by the following equations:

$$\text{Probability } P_L < 20\% \quad MSF = 10^{3.81}/M_w^{4.53} \text{ for } M_w < 7 \quad (27)$$

$$\text{Probability } P_L < 32\% \quad MSF = 10^{3.74}/M_w^{4.33} \text{ for } M_w < 7 \quad (28)$$

$$\text{Probability } P_L < 50\% \quad MSF = 10^{4.21}/M_w^{4.81} \text{ for } M_w < 7.75 \quad (29)$$

### New Recommendation by Idriss

I. M. Idriss (TRB 1999) proposed a new set of MSFs that are compatible with, and are only to be used with, the magnitude-dependent  $r_d$  that he also proposed. These new MSFs have lower values than the revised MSFs listed in Table 3, but slightly higher values than the original Seed and Idriss (1982) MSFs. Because the proposed  $r_d$  and associated MSFs have not been published and the factors have not been independently verified, the workshop participants chose not to recommend the new  $r_d$  or MSFs at this time.

### Recommendations for Engineering Practice

The workshop participants reviewed the MSFs listed in Table 3, and all but one (S. S. C. Liao) agree that the original factors were too conservative and that increased MSFs are warranted for engineering practice for magnitudes <7.5. Rather than recommending a single set of factors, the workshop participants suggest a range of MSFs from which the engineer is allowed to choose factors that are requisite with the acceptable risk for any given application. For magnitudes <7.5, the lower bound for the recommended range is the new MSF proposed by Idriss [column 3 in Table 3, or (23)]. The suggested upper bound is the MSF proposed by Andrus and Stokoe [column 7 in Table 3, or (26)]. The upper-bound values are consistent with MSFs suggested by Ambraseys (1988), Arango (1996), and Youd and Noble (1997a) for  $P_L < 20\%$ .

For magnitudes >7.5, the new factors recommended by Idriss [column 3 in Table 3; (25)] should be used for engineering practice. These new factors are smaller than the original Seed and Idriss (1982) factors, hence their application leads to increased calculated liquefaction hazard compared to the original factors. Because there are only a few well-documented liquefaction case histories for earthquakes with magnitudes >8, MSFs in that range are poorly constrained by field data. Thus the workshop participants agreed that the greater conservatism embodied in the revised MSF by Idriss (column 3, Table 3) should be recommended for engineering practice.

### CORRECTIONS FOR HIGH OVERBURDEN STRESSES, STATIC SHEAR STRESSES, AND AGE OF DEPOSIT

Correction factors  $K_o$  and  $K_a$  were developed by Seed (1983) to extrapolate the simplified procedure to larger overburden pressure and static shear stress conditions than those embodied in the case history data set from which the simplified procedure was derived. As noted previously, the simplified procedure was developed and validated only for level to gently sloping sites (low static shear stress) and depths less than about 15 m (low overburden pressures). Thus applications using  $K_o$  and  $K_a$  are beyond routine practice and require specialized expertise. Because these factors were discussed at the workshop and some new information was developed, recommendations from those discussions are included here. These rec-

ommendations, however, apply mostly to liquefaction hazard analyses of embankment dams and other large structures. These factors are applied by extending (23) to include  $K_\sigma$  and  $K_\alpha$  as follows:

$$FS = (CRR_{7.5}/CSR) \cdot MSF \cdot K_\sigma \cdot K_\alpha \quad (30)$$

### $K_\sigma$ Correction Factor

Cyclically loaded laboratory test data indicate that liquefaction resistance increases with increasing confining stress. The rate of increase, however, is nonlinear. To account for the non-linearity between CRR and effective overburden pressure, Seed (1983) introduced the correction factor  $K_\sigma$  to extrapolate the simplified procedure to soil layers with overburden pressures  $>100$  kPa. Cyclically loaded, isotropically consolidated triaxial compression tests on sand specimens were used to measure CRR for high-stress conditions and develop  $K_\sigma$  values. By taking the ratio of CRR for various confining pressures to the CRR determined for approximately 100 kPa (1 atm) Seed (1983) developed the original  $K_\sigma$  correction curve. Other investigators have added data and suggested modifications to better define  $K_\sigma$  for engineering practice. For example, Seed and Harder (1990) developed the clean-sand curve reproduced in Fig. 13. Hynes and Olsen (1999) compiled and analyzed an enlarged data set to provide guidance and formulate equations for selecting  $K_\sigma$  values (Fig. 14). The equation they derived for calculating  $K_\sigma$  is

$$K_\sigma = (\sigma'_{vo}/P_a)^{f-1} \quad (31)$$

where  $\sigma'_{vo}$ , effective overburden pressure; and  $P_a$ , atmospheric pressure, are measured in the same units; and  $f$  is an exponent that is a function of site conditions, including relative density, stress history, aging, and overconsolidation ratio. The workshop participants considered the work of previous investigators and recommend the following values for  $f$  (Fig. 15). For relative densities between 40 and 60%,  $f = 0.7-0.8$ ; for relative densities between 60 and 80%,  $f = 0.6-0.7$ . Hynes and Olsen recommended these values as minimal or conservative estimates of  $K_\sigma$  for use in engineering practice for both clean and silty sands, and for gravels. The workshop participants concurred with this recommendation.

### $K_\alpha$ Correction Factor for Sloping Ground

The liquefaction resistance of dilative soils (moderately dense to dense granular materials under low confining stress) increases with increased static shear stress. Conversely, the liquefaction resistance of contractive soils (loose soils and moderately dense soils under high confining stress) decreases with increased static shear stresses. To incorporate the effect of static shear stresses on liquefaction resistance, Seed (1983) introduced a correction factor  $K_\alpha$ . To generate values for this factor, Seed normalized the static shear stress  $\tau_{st}$  acting on a plane with respect to the effective vertical stress  $\sigma'_{vo}$  yielding a parameter  $\alpha$ , where

$$\alpha = \tau_{st}/\sigma'_{vo} \quad (32)$$

Cyclically loaded triaxial compression tests were then used to empirically determine values of the correction factor  $K_\alpha$  as a function of  $\alpha$ .

For the NCEER workshop, Harder and Boulanger (1997) reviewed past publications, test results, and analyses of  $K_\alpha$ . They noted that a wide range of  $K_\alpha$  values have been proposed, indicating a lack of convergence and a need for continued research. The workshop participants agreed with this assessment. Although curves relating  $K_\alpha$  to  $\alpha$  have been published (Harder and Boulanger 1997), these curves should not be used

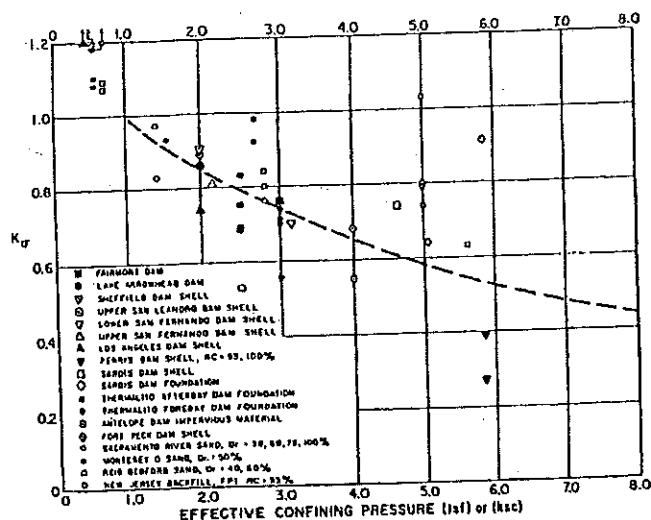


FIG. 13.  $K_\sigma$ -Values Determined by Various Investigators (Reproduced from Seed and Harder 1990)

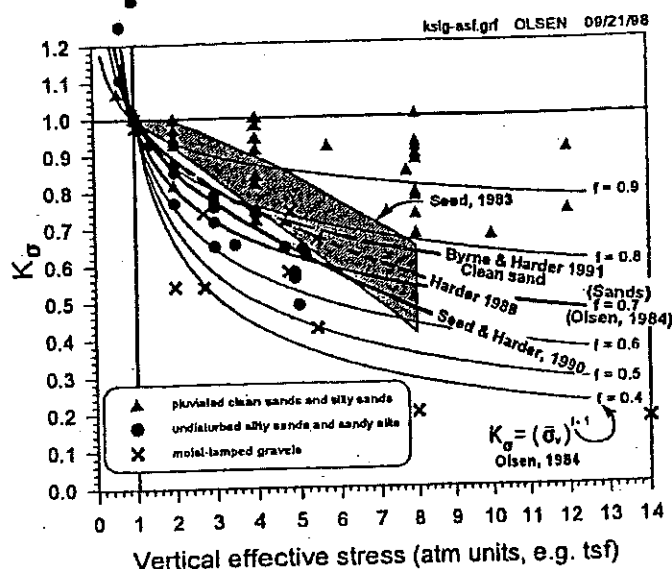


FIG. 14. Laboratory Data and Compiled  $K_\sigma$  Curves (Reproduced from Hynes and Olsen 1999)

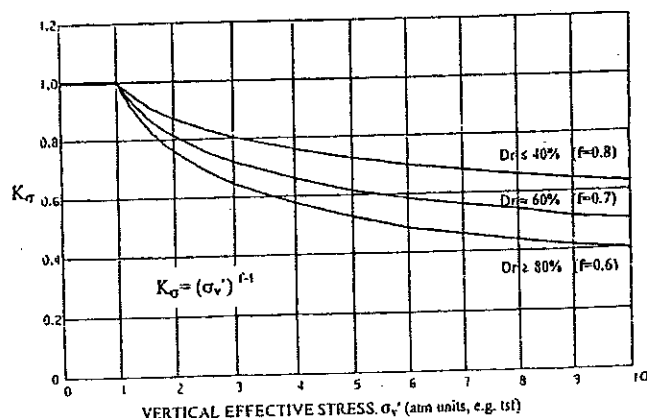


FIG. 15. Recommended Curves for Estimating  $K_\sigma$  for Engineering Practice

by nonspecialists in geotechnical earthquake engineering or in routine engineering practice.

### Influence of Age of Deposit

Several investigators have noted that liquefaction resistance of soils increases with age. For example, Seed (1979) observed significant increases in liquefaction resistance with aging of reconstituted sand specimens tested in the laboratory. Increases of as much as 25% in cyclic resistance ratio were noted between freshly constituted and 100-day-old specimens. Youd and Hoose (1977) and Youd and Perkins (1978) noted that liquefaction resistance increases markedly with geologic age. Sediments deposited within the past few thousand years are generally much more susceptible to liquefaction than older Holocene sediments; Pleistocene sediments are even more resistant; and pre-Pleistocene sediments are generally immune to liquefaction. Although qualitative time-dependent increases have been documented as noted above, few quantitative data have been collected. In addition, the factors causing increased liquefaction resistance with age are poorly understood. Consequently, verified correction factors for age have not been developed.

In the absence of quantitative correction factors, engineering judgment is required to estimate the liquefaction resistance of sediments more than a few thousand years old. For deeply buried sediments dated as more than a few thousand years old, some knowledgeable engineers have omitted application of the  $K_s$  factor as partial compensation for the unquantified, but substantial increase of liquefaction resistance with age. For man-made structures, such as thick fills and embankment dams, aging effects are minimal, and corrections for age should not be applied in calculating liquefaction resistance.

### SEISMIC FACTORS

Application of the simplified procedure for evaluating liquefaction resistance requires estimates of two ground motion parameters—earthquake magnitude and peak horizontal ground acceleration. These factors characterize duration and intensity of ground shaking, respectively. The workshop addressed the following questions with respect to selection of magnitude and peak acceleration values for liquefaction resistance analyses.

### Earthquake Magnitude

Records from recent earthquakes, such as 1979 Imperial Valley, 1988 Armenia, 1989 Loma Prieta, 1994 Northridge, and 1995 Kobe, indicate that the relationship between duration and magnitude is rather uncertain and that factors other than magnitude also influence duration. For example, unilateral faulting, in which rupture begins at one end of the fault and propagates to the other, usually produces longer shaking duration for a given magnitude than bilateral faulting, in which slip begins near the midpoint on the fault and propagates in both directions simultaneously. Duration also generally increases with distance from the seismic energy source and may vary with tectonic province, site conditions, and bedrock topography (basin effects).

**Question:** Should correction factors be developed to adjust duration of shaking to account for the influence of earthquake source mechanism, fault rupture mode, distance from the energy source, basin effects, etc.?

**Answer:** Faulting characteristics and variations in shaking duration are difficult to predict in advance of an earthquake event. The influence of distance generally is of secondary importance within the range of distances to which damaging liq-

uefaction effects commonly develop. Basin effects are not yet sufficiently predictable to be adequately accounted for in engineering practice. Thus the workshop participants recommend continued use of the generally conservative relationship between magnitude and duration that is embodied in the simplified procedure.

**Question:** An important difference between eastern U.S. earthquakes and western U.S. earthquakes is that eastern ground motions are generally richer in high-frequency energy and thus could generate more significant stress cycles and equivalently longer durations than western earthquakes of the same magnitude. Is a correction needed to account for higher frequencies of motions generated by eastern U.S. earthquakes?

**Answer:** The high-frequency motions of eastern earthquakes are generally limited to near-field rock sites. High-frequency motions attenuate or are damped out rather quickly as they propagate through soil layers. This filtering action reduces the high-frequency energy at soil sites and thus reduces differences in numbers of significant loading cycles. Because liquefaction occurs only within soil strata, duration differences on soil sites between eastern and western earthquakes are not likely to be great. Without more instrumentally recorded data from which differences in ground motion characteristics can be quantified, there is little basis for the development of additional correction factors for eastern localities.

Another difference between eastern and western U.S. earthquakes is that strong ground motions generally propagate to greater distances in the east than in the west. By applying present state-of-the-art procedures for estimating peak ground acceleration at eastern sites, differences in amplitudes of ground motions between western and eastern earthquakes are properly taken into account.

**Question:** Which magnitude scale should be used for selection of earthquake magnitudes for liquefaction resistance analyses?

**Answer:** Seismologists commonly calculate earthquake magnitudes using five different scales: (1) local or Richter magnitude  $M_L$ ; (2) surface-wave magnitude  $M_s$ ; (3) short-period body-wave magnitude  $m_b$ ; (4) long-period body-wave magnitude  $m_B$ ; and (5) moment magnitude  $M_w$ . Moment magnitude, the scale most commonly used for engineering applications, is the scale preferred for calculation of liquefaction

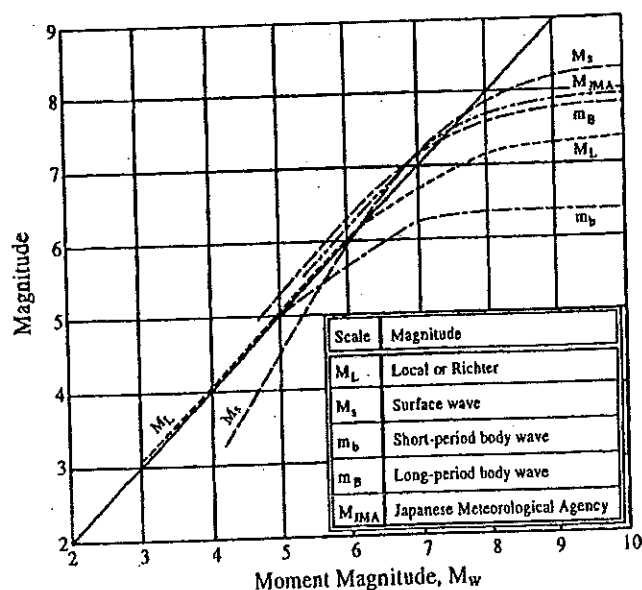


FIG. 16. Relationship between Moment  $M_w$  and Other Magnitude Scales (Reproduced from Heaton et al. Unpublished Report, 1982)

resistance. As Fig. 16 shows, magnitudes from other scales may be substituted directly for  $M_w$  within the following limitations— $M_L < 6$ ,  $m_B < 7.5$ , and  $6 < M_L < 8$ — $m_b$ , a scale commonly used for eastern U.S. earthquakes, may be used for magnitudes between 5 and 6, provided  $m_b$  values are corrected to equivalent  $M_w$  values. The curves plotted in Fig. 16 may be used for this adjustment (Idriss 1985).

## Peak Acceleration

In the simplified procedure, peak horizontal acceleration  $a_{max}$  is used to characterize the intensity of ground shaking. To provide guidance for estimation of  $a_{max}$ , the workshop addressed the following questions.

**Question:** What procedures are preferred for estimating  $a_{max}$  at potentially liquefiable sites?

**Answer:** The following methods, in order of preference, may be used for estimating  $a_{max}$ :

1) The preferred method for estimating  $a_{max}$  is through empirical correlations of  $a_{max}$  with earthquake magnitude, distance from the seismic energy source, and local site conditions. Several correlations have been published for estimating  $a_{max}$  for sites on bedrock or stiff to moderately stiff soils. Preliminary attenuation relationships have also been developed for a limited range of soft soil sites (Idriss 1991). Selection of an attenuation relationship should be based on such factors as region of the country, type of faulting, and site condition.

2) For soft sites and other soil profiles that are not compatible with available attenuation relationships,  $a_{max}$  may be estimated from local site response analyses. Computer programs such as SHAKE and DESRA may be used for these calculations (Schnabel et al. 1972; Finn et al. 1977). Input ground motions in the form of recorded accelerograms are preferable to synthetic records. Accelerograms derived from white noise should be avoided. A suite of plausible earthquake records should be used in the analysis, including as many as feasible from earthquakes with similar magnitudes, source distances, etc.

3) The third and least desirable method for estimating peak ground acceleration is through amplification ratios, such as those developed by Idriss (1990, 1991) and Seed et al. (1994). These factors use a multiplier or ratio by which bedrock outcrop motions are amplified to estimate surface motions at soil sites. Because amplification ratios are influenced by strain level, earthquake magnitude, and frequency content, caution and considerable engineering judgment are required in the application of these relationships.

**Question:** Which peak acceleration should be used: (1) the largest horizontal acceleration recorded on a three-component accelerogram; (2) the geometric mean (square root of the product) of the two maximum horizontal components; or (3) a vectorial combination of horizontal accelerations?

**Answer:** According to I. M. Idriss (oral discussion at NCEER workshop, 1996), where recorded motions were available, the larger of the two horizontal peak components of acceleration was used in the compilation of data used to derive the original simplified procedure. Where recorded values were not available, which was the circumstance for most sites, peak acceleration values were estimated from attenuation relationships based on the geometric mean of the two orthogonal peak horizontal accelerations. In nearly all instances where recorded motions were used, the peaks from the two horizontal records were approximately equal. Thus where a single peak was used, the peak and the geometric mean of the two peaks were about the same value. Based on this information, the workshop participants concurred that use of the geometric mean is consistent with the development of the procedure and is preferred for use in engineering practice. However, use of the larger of the two orthogonal peak accelerations yields a larger estimate

of  $a_{max}$ , is conservative, and is allowable. Vectorial accelerations are seldom calculated and should not be used. Peak vertical accelerations are generally much smaller than peak horizontal accelerations and are ignored for calculation of liquefaction resistance.

**Question:** Liquefaction usually develops at soil sites where ground motion amplification may occur and where sediment may soften, reducing motions as excess pore pressure develops. How should investigators account for these factors in estimating peak acceleration?

**Answer:** The recommended procedure is to calculate or estimate the  $a_{max}$  that would occur at the site in the absence of increased pore pressure or the onset of liquefaction. That peak acceleration incorporates the influence of site amplification, but neglects the influence of excess pore-water pressure.

**Question:** Should high-frequency spikes (periods  $< 0.1$  s) in acceleration records be considered or ignored?

**Answer:** In general, short-duration, high-frequency acceleration spikes are too short in duration to generate significant instability or deformation of granular structures, and should be ignored. By using attenuation relationships for estimation of peak acceleration, as noted above, high-frequency spikes are essentially ignored because few high-frequency peaks are incorporated in databases from which attenuation relationships were derived. Similarly, ground response analyses programs such as SHAKE and DESRA generally attenuate or filter out high-frequency spikes, reducing their influence. Where amplification ratios are used, engineering judgment should be used to determine which bedrock acceleration is to be amplified.

## ENERGY-BASED CRITERIA AND PROBABILISTIC ANALYSES

The workshop considered two additional topics: (1) liquefaction resistance criteria based on seismic energy passing through a liquefiable layer (Kayen and Mitchell 1997; Youd et al. 1997), and probabilistic analyses of case history data (Liao et al. 1988; Youd and Noble 1997b). Although probabilistic or risk analyses have been made for some localities and critical facilities, the workshop participants concluded that probabilistic procedures are still under development and not sufficiently formulated for routine engineering practice. Similarly, new energy-based criteria need to be independently tested before recommendations can be made for general practice. The workshop participants recommend that research and development continue on both of these relatively new and potentially useful procedures.

## CONCLUSIONS

The participants in the NCEER workshop reviewed the state-of-the-art for evaluating liquefaction resistance and recommend several augmentations to that procedure. Specific recommendations, including procedures and equations, are listed in each section of this summary paper. Consensus conclusions from the workshop are:

1. Four field tests are recommended for routine evaluation of liquefaction resistance—the cone penetration test (CPT), the standard penetration test (SPT), shear-wave velocity ( $V_s$ ) measurements, and for gravelly sites the Becker penetration test (BPT). Criteria for each test were reviewed and revised to incorporate recent developments and to achieve consistency between resistances calculated from the various tests. Each test has its advantages and limitations (Table 1). The CPT provides the most detailed soil stratigraphy and robust field-data based liquefaction resistance curves now available. CPT testing



should always be accompanied by soil sampling for validation of soil type identification. The SPT has a longer record of application and provides disturbed soil samples from which fines content and other grain characteristics can be determined. Measured shear-wave velocities provide fundamental information on small-strain soil behavior that is useful beyond analyses of liquefaction resistance.  $V_s$  is also applicable at sites, such as landfills and gravelly sediments, where CPT and SPT soundings may not be possible or reliable. The BPT test is recommended only for gravelly sites and requires use of rough correlations between BPT and SPT, making the results less certain than other tests. Where possible, two or more test procedures should be applied to assure adequate definition of soil stratigraphy and a consistent evaluation of liquefaction resistance.

2. The magnitude scaling factors originally derived by Seed and Idriss (1982) are overly conservative for earthquakes with magnitudes  $<7.5$ . A range of scaling factors is recommended for engineering practice, the lower end of the range being the new MSF recommended by Idriss (column 3, Table 3), and the upper end of the range being the MSF suggested by Andrus and Stokoe (column 7, Table 3). These MSFs are defined by (25) and (26), respectively. For magnitudes  $>7.5$ , the new factors by Idriss (column 3, Table 3) should be used. These factors, which are more conservative than the original Seed and Idriss (1982) factors, should be applied.
3. The  $K_a$  factors suggested by Seed and Harder (1990) appear to be overly conservative for some soils and field conditions. The workshop participants recommend  $K_a$  values defined by the curves in Fig. 14 or (31). Because  $K_a$  values are usually applied to depths greater than those verified for the simplified procedure, special expertise is generally required for their application.
4. Procedures for evaluation of liquefaction resistance beneath sloping ground or embankments (slopes greater than about 6%) have not been developed to a level allowable for routine use. Special expertise is required for evaluation of liquefaction resistance beneath sloping ground.
5. Moment magnitude  $M_w$  should be used for liquefaction resistance calculations. Magnitude, as used in the simplified procedure, is a measure of the duration of strong ground shaking. The present magnitude criteria are conservative and should not be corrected for source mechanism, style of faulting, distance from the energy source, subsurface bedrock topography (basin effect), or tectonic region (eastern versus western U.S. earthquakes).
6. The peak acceleration  $a_{max}$  applied in the procedure is the peak horizontal acceleration that would occur at ground surface in the absence of pore pressure increases or liquefaction. Attenuation relationships compatible with soil conditions at a site should be applied in estimating  $a_{max}$ . Relationships based on the geometric mean of the peak horizontal accelerations are preferred, but use of relationships based on peak horizontal acceleration is allowable and conservative. Where site conditions are incompatible with existing attenuation relationships, site-specific response calculations, using programs such as SHAKE or DESRA, should be used. The least preferable technique is application of amplification factors.

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## WORKSHOP PARTICIPANTS

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## APPENDIX II. NOTATION

The following symbols are used in this paper:

- $a, b$  = curve fitting parameters for use with  $V_s$  criteria for evaluating liquefaction resistance;
- $a_{max}$  = peak horizontal acceleration at ground surface;
- $C_B$  = correction factor for borehole diameter;
- $C_E$  = correction factor for hammer energy;
- $C_N$  = correction factor for overburden pressure applied to SPT;
- $C_Q$  = correction factor for overburden pressure applied to CPT;
- $C_R$  = correction factor for drilling rod length;
- $C_S$  = correction factor for split spoon sampler without liners;
- $CRR_{7.5}$  = cyclic resistance ratio for  $M_w = 7.5$  earthquakes;
- $d_c$  = diameter of CPT tip;
- $F$  = normalized friction ratio;
- $f$  = exponent estimated from site conditions used in calculation of  $K_{\sigma}$ ;
- $f_s$  = sleeve friction measured with CPT;
- $g$  = acceleration of gravity;
- $H$  = thickness of thin granular layer between softer sediment layers;
- $I_c$  = soil behavior type index for use with CPT liquefaction criteria;



$K_c$  = correction factor for grain characteristics applied to CPT;  
 $K_H$  = thin-layer correction factor for use with CPT;  
 $K_s$  = correction factor for soil layers subjected to large static shear stresses;  
 $K_n$  = correction factor for soil layers subjected to large static normal stresses;  
 $M_L$  = local or Richter magnitude of earthquake;  
 $M_s$  = surface-wave magnitude of earthquake;  
 $M_w$  = moment magnitude of earthquake;  
 $m_B$  = long period body-wave magnitude of earthquake;  
 $m_b$  = short period body-wave magnitude of earthquake;  
 $N_m$  = measured standard penetration resistance;  
 $(N_1)_{60}$  = corrected standard penetration resistance;  
 $(N_1)_{60cr}$  =  $(N_1)_{60}$  adjusted to equivalent clean-sand value;  
 $n$  = exponent used in normalizing CPT resistance for overburden stress;  
 $P_a$  = atmospheric pressure, approximately 100 kPa;  
 $P_L$  = probability of liquefaction;

$Q$  = normalized and dimensionless cone penetration resistance;  
 $q_{c1N}$  = normalized cone penetration resistance;  
 $(q_{c1N})_{cr}$  = normalized cone penetration resistance adjusted to equivalent clean-sand value;  
 $r_d$  = stress reduction coefficient to account for flexibility in soil profile;  
 $V_s$  = measured shear-wave velocity;  
 $V_{s1}$  = overburden-stress corrected shear-wave velocity;  
 $V_{s1}^*$  = limiting upper value of  $V_{s1}$  for liquefaction occurrences;  
 $z$  = depth below ground surface (m);  
 $\alpha, \beta$  = coefficients, that are functions of fines content, used to correct  $(N_1)_{60}$  to  $(N_1)_{60cr}$ ;  
 $\sigma'_{vo}$  = effective overburden pressure;  
 $\tau_{av}$  = average horizontal shear stress acting on soil layer during shaking generated by given earthquake; and  
 $\tau_{st}$  = static shear stress acting on soil element due to gravitational forces.

## Appendix C

### Application of $V_s$ for Estimating Sand Compressibility



# THE SEISMIC CPT METHOD FOR ESTIMATING LIQUEFACTION PARAMETERS

(NCEER Workshop, 1998)

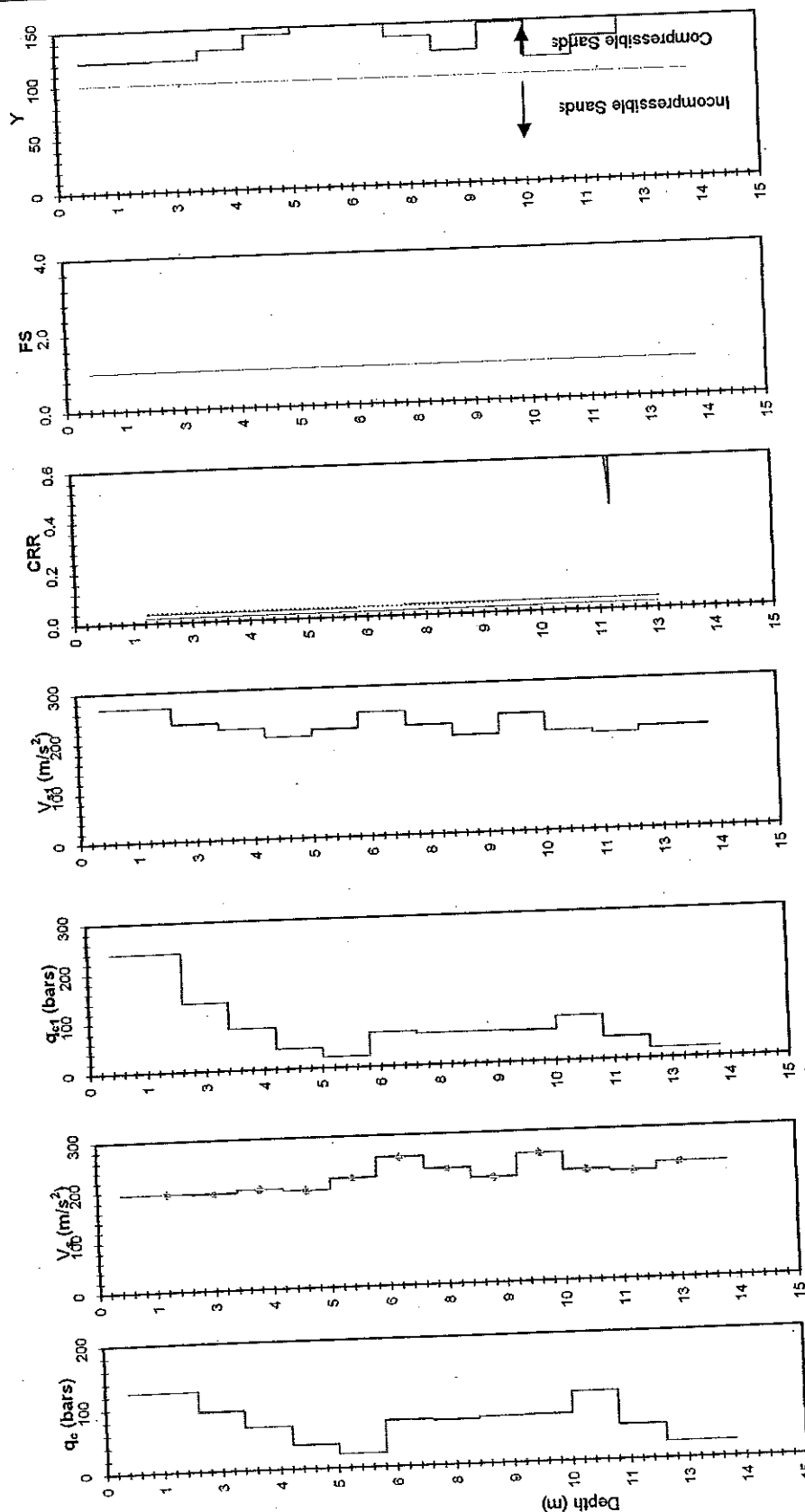
Date: 05:17:04 12:50 CONETEC

Hole

WT: 5.70m

CPT: Location: HWY69 4L ESTAIRE 20 Ton St 113 SCPT04-01S

Depth: 12.50m



FS = 1.0  
max a = 0.05  
min a = 0.1

CSR mag = 6.00  
max a = 0.05  
min a = 0.1

Y = 100

CPT data can be useful for providing a preliminary estimate of liquefaction potential in sandy soils when used in conjunction with shear wave velocity.



# THE SEISMIC CPT METHOD FOR ESTIMATING LIQUEFACTION PARAMETERS

(NCEER Workshop, 1998)

Hole

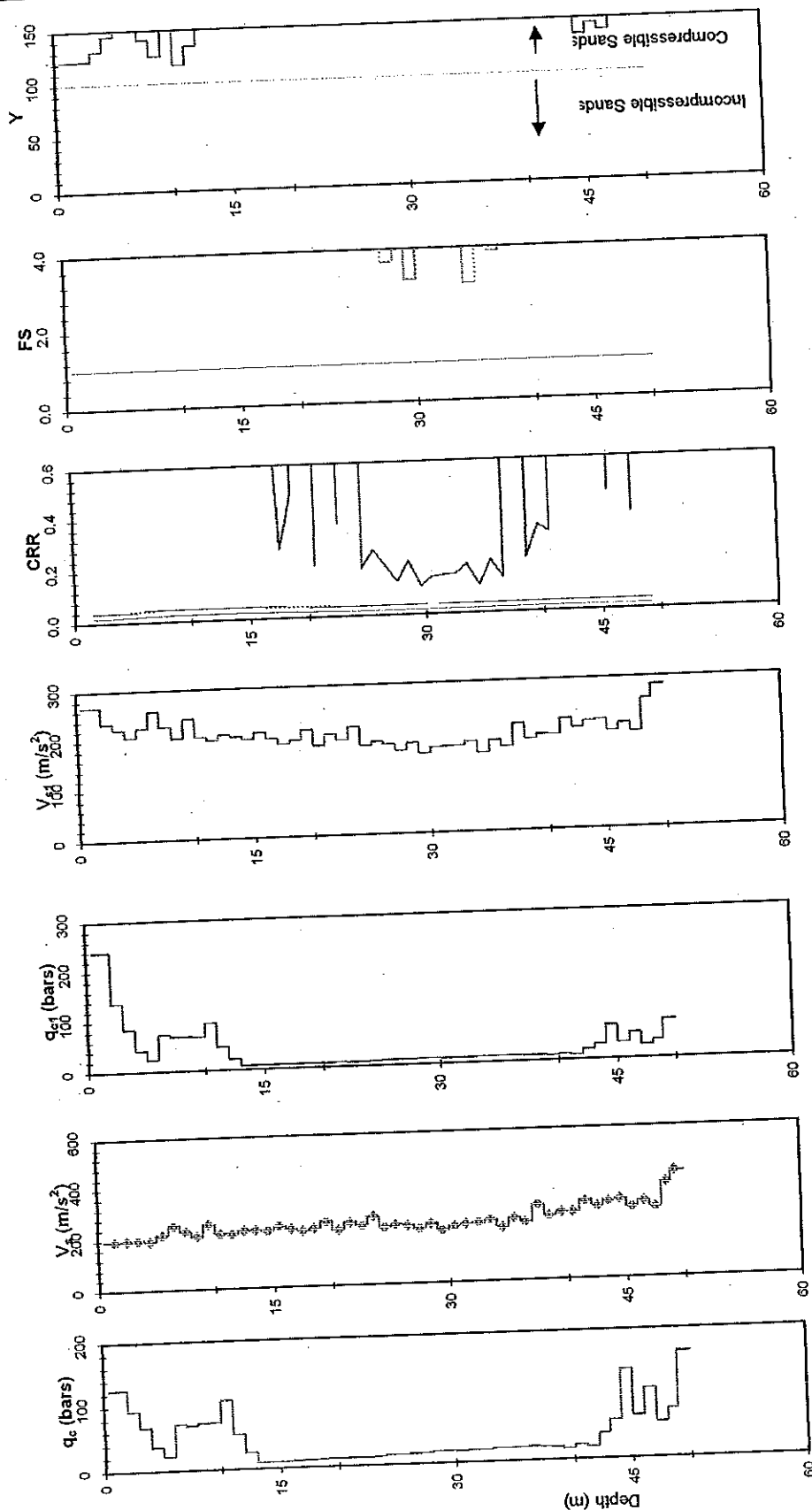
Date: 05:17:04 12:50 CONETEC

WT: 4.00m

SCPT04-01S

Depth: 49.25m

CPT: Location: HWY69 4L ESTAIRE 20 Ton St 113



CSR mag = 6.00  
max a = 0.05  
max a = 0.1

FS = 1.0  
max a = 0.05  
max a = 0.1

Y = 100

# INTEGRATED SEISMIC CPT METHOD FOR EVALUATING CYCLIC RESISTANCE RATIO (CRR)

(NCEER Workshop, 1998)

CPT data can be useful for providing a preliminary estimate of liquefaction potential in sandy soils when used in conjunction with shear wave velocity.

Legend: XXXXXXXXXX Factor of Safety < 1.  
XXXXXXXXXX Liquefaction potential high.

File Data  
 File Name  
 Location  
 Cone

Date  
 20 Ton St 11: Time  
 Operator:

ETEC

Site Parameters  
 Water Table  
 Unit Weight of Water  
 Atmospheric Pressure

Earthquake Parameters  
 Earthquake Magnitude (typ 7.5)  
 max acceleration 1  
 max acceleration 2  
 Maximum shear wave velocity

Depth (m)	q <sub>c</sub> (bars)	V <sub>s</sub> (m/s <sup>2</sup> )	γ (kN/m <sup>3</sup> )	σ <sub>VT</sub> (kN/m <sup>2</sup> )	σ <sub>vo</sub> <sup>1</sup> (kN/m <sup>2</sup> )	q <sub>c1</sub> (bars)	V <sub>s1</sub> (m/s <sup>2</sup> )	CRR	CSR <sub>7.5</sub> a max 1	CSR <sub>7.5</sub> a max 2	CSR <sub>var</sub> a max 1	CSR <sub>var</sub> a max 2	FS a max 1	FS a max 2	Y
1.50	125.91	195.00	18.50	27.75	27.75	239.01	268.67	1000.00	0.03	0.06	0.02	0.04	1000.00	1000.00	121.51
2.50	93.42	193.95	18.50	46.25	46.25	137.36	235.18	1000.00	0.03	0.06	0.02	0.04	1000.00	1000.00	122.16
3.50	67.66	199.71	18.50	64.75	64.75	84.09	222.63	1000.00	0.03	0.06	0.02	0.04	1000.00	1000.00	130.74
4.50	37.19	194.58	18.50	83.25	83.25	40.76	203.70	1000.00	0.03	0.06	0.02	0.04	1000.00	1000.00	143.36
5.50	22.17	217.03	18.50	101.75	101.75	21.98	216.09	1000.00	0.03	0.06	0.02	0.04	1000.00	1000.00	177.48
6.50	71.99	255.71	18.50	120.25	112.40	67.91	248.34	1000.00	0.03	0.07	0.02	0.04	1000.00	1000.00	153.84
7.50	69.61	229.94	18.50	138.75	121.09	63.25	219.20	1000.00	0.04	0.07	0.02	0.04	1000.00	1000.00	138.22
8.50	72.49	209.31	18.50	157.25	129.78	63.63	196.10	0.79	0.04	0.07	0.02	0.04	37.76	18.88	123.47
9.50	73.78	254.78	18.50	175.75	138.47	62.70	234.86	1000.00	0.04	0.08	0.02	0.04	1000.00	1000.00	148.42
10.50	107.62	219.07	18.50	194.25	147.16	88.72	198.90	2.62	0.04	0.08	0.02	0.04	120.20	60.10	115.25
11.50	54.09	214.41	18.50	212.75	155.85	43.32	191.89	0.41	0.04	0.08	0.02	0.04	18.89	9.44	133.01
12.50	25.51	229.72	18.50	231.25	164.54	19.89	202.82	1000.00	0.04	0.08	0.02	0.04	1000.00	1000.00	170.79

## ESTIMATION OF SAND COMPRESSIBILITY FROM SEISMIC CPT

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### ABSTRACT

The interpretation of penetration tests in cohesionless soils for geotechnical parameters, such as relative density, in-situ state (state parameter) and friction angle, is sensitive to soil compressibility. More than a decade of large calibration chamber testing has clearly illustrated the importance of soil compressibility on penetration resistance. Highly compressible sands can have a very low penetration resistance compared to relatively incompressible sands at the same void ratio and stress level. A low measured penetration resistance in compressible sands can lead to an incorrect interpretation of a very loose material. Silty sands tend to be more compressible than clean sands, however, corrections based on percent fines content are not always valid. Recent research has illustrated the potential of shear wave velocity to evaluate the in-situ state of young, uncemented sands. The combined measurement of shear wave velocity and penetration resistance using the seismic cone penetration test (SCPT) can be used to evaluate the compressibility of cohesionless soils. This paper presents laboratory and field evidence to illustrate the proposed interpretation. The limitations of the proposed technique are also discussed.

### INTRODUCTION

The Cone Penetration Test (CPT) has become increasingly more popular due to the continuous nature of the data, reliable and repeatable results and cost effectiveness. The primary application of the CPT is for stratigraphic profiling, estimation of geotechnical parameters and to provide

results for direct geotechnical design. Empirical and theoretical correlations exist relating cone penetration resistance to a full range of geotechnical parameters.

Some of the most significant research in the last decade on the interpretation of cone penetration resistance in sands has been carried out using large calibration chambers. This research has clearly shown that a relationship exists between CPT penetration resistance ( $q_c$ ) and sand relative density ( $D_r$ ). However, this relationship is not unique for all sands and is strongly influenced by the compressibility of a given sand.

Highly compressible sands, such as sands with a high mica or carbonate content or high fines content, can have a very low penetration resistance compared to a relatively incompressible, rounded, clean silica sand at the same density and stress level. A low penetration resistance in highly compressible sands can lead to an incorrect interpretation of a very loose material. For design issues such as pile design or liquefaction evaluation, the low penetration resistance may lead to overly conservative designs. For liquefaction evaluation current methods allow for a correction based on fines content. The penetration resistance in a silty sand can be corrected upwards to an equivalent clean sand value based on the percentage fines content. This type of correction essentially compensates for the increased compressibility of the silty sand. However, the percent fines content is not always a good measure of the soil compressibility, since some clean sands can have very high compressibility even though the fines content is low.

To improve the interpretation of the CPT in sands it is important to estimate the sand compressibility. However, there has been very little guidance on how to estimate soil compressibility without the need for samples and laboratory testing.

This paper investigates the potential of using the combined independent measurements of penetration resistance and shear wave velocity to estimate soil compressibility.

## EVALUATION OF IN-SITU STATE

Wroth and Bassett (1965) recognized that sand density was not a good indicator of sand state since the response of a sand is controlled by both the density and the stress level. Been and Jefferies (1986) suggested the concept of state parameter ( $\psi$ ) to combine the effects of density and stress to define the state of a sand. The state parameter was defined as the difference in void ratio between the current void ratio ( $e$ ) of the sand and the void ratio at ultimate steady state ( $e_{ss}$ ) at the same stress level. Based on a review of large calibration chamber test results, Been et al., (1986) proposed a method to estimate the state of a sand from CPT penetration resistance. However, there is no unique correlation between normalized cone penetration resistance and state parameter for all sands. The major disadvantage with this method has been the need to carry out testing in a large calibration chamber to derive the specific correlation for a given sand. This can be extremely expensive and time consuming and has generally been reserved for only very large projects. There has also been some uncertainty over corrections to the CPT results due to the limited size of the calibration chambers.

Been et al., (1986) suggested that the slope of the steady state line ( $\lambda$ ) was an importance measure to identify the type of sand. Recent research (Fear and Robertson, 1995) has shown that the slope of the steady state line is controlled by the grain characteristics and is indirectly related to the sand compressibility. Sands with a high compressibility tend to have a steep steady state line (i.e. high value a  $\lambda$ ). Been and Jefferies (1992) have extended their method to include sands and clays and use  $\lambda$  as a key parameter. However, the application of this method requires an estimate of the slope of the steady state line for the soil and hence some laboratory testing.

Robertson et al. (1994) recently suggested a method to estimate the state of a young, uncemented sand using shear wave velocity ( $V_s$ ). This method links the measurement of  $V_s$  on small samples in the laboratory at a known state with in-situ measurements of  $V_s$ . The shear wave velocity is primarily a function of soil void ratio and effective confining stresses as well as soil age, cementation and fabric. For young, uncemented sands the shear wave velocity is dominated by the void ratio and effective stresses. Hence, the potential exists to estimate the in-situ state of a young, uncemented sand from in-situ shear wave velocity measurements. However, this method still requires a knowledge of the location and slope of the steady state line. Figure 1 shows the relationship between normalized shear wave velocity ( $V_{s1}$ ) and void ratio ( $e$ ) for a range of freshly deposited uncemented sand samples in the laboratory. The normalized shear wave velocity ( $V_{s1}$ ) is defined as follows;

$$(1) \quad V_{s1} = V_s (Pa/\sigma_{vo}')^{0.25}$$

where:  $V_s$  = shear wave velocity  
 $Pa$  = atmospheric pressure, usually = 100 kPa  
 $\sigma_{vo}'$  = vertical effective stress.

When the relationship shown in Figure 1 is combined with the location of the steady state line it is possible to estimate the in-situ state of a sand based on in-situ measurements of shear wave velocity (Robertson et al., 1994). The in-situ shear wave velocity is controlled by the effective stresses in both the direction of wave propagation and the direction of particle motion. For the seismic CPT this means that  $V_s$  is controlled by the vertical and horizontal effective stresses. Hence, equation 1 should include the horizontal stress as well as the vertical stress. However, in practice the horizontal stress is generally unknown therefore, the recommended normalization includes only the vertical effective stress. The error in excluding the horizontal effective stress is generally less than 10%. However, when combining the in-situ shear wave velocity measurements with the steady state line it is advisable to account for  $K_o$  conditions.

The relationship shown in Figure 1 is based on testing 7 different sands. Even though these sands had very different grain characteristics and compressibility's, as reflected in their values of  $\lambda$ , the normalized shear wave velocity correlation with void ratio appears to be little influenced. This is consistent, since the shear wave velocity is a very small strain measurement which would not be expected to be influenced by the large strain compressibility. Robertson et al.(1994) suggested that soil fabric appears to have only a small influence on the correlation between normalized shear wave velocity and void ratio, as supported by the results in Figure 1.



Since shear wave velocity is little influenced by sand compressibility, while the cone penetration resistance is strongly influenced by sand compressibility, there appears to be the potential to identify sand compressibility by comparing shear wave velocity and cone penetration resistance in a sand at the same depth. This can be accomplished using the seismic CPT in which both the cone resistance and shear wave velocity are measured during the same sounding in the same soil.

## ESTIMATION OF SAND COMPRESSIBILITY

Since cone penetration resistance and shear wave velocity vary with overburden effective stresses in different ways, it is important to compare normalized values. The cone penetration resistance can be normalized as follows:

$$(2) \quad q_{c1} = q_c (P_a / \sigma_{vo}')^{0.5}$$

where:  $q_{c1}$  = normalized cone penetration resistance  
 $P_a$  = atmospheric pressure, usually = 100 kPa  
 $\sigma_{vo}'$  = vertical effective stress.

The relationship between shear wave velocity and cone penetration resistance can be given by the following;

$$(3) \quad q_{c1} = (V_{s1} / Y)^4$$

where:  $q_{c1}$  is in MPa  
 $V_{s1}$  is in m/s.

The parameter Y appears to be controlled by grain characteristics, sand compressibility, age and degree of cementation. For more compressible sands the parameter Y will increase since the normalized cone penetration resistance  $q_{c1}$  will decrease while the normalized shear wave velocity ( $V_{s1}$ ) remains essentially constant. For aged or cemented sands Y will also increase since the shear wave velocity will increase faster than the penetration resistance.

As part of the CANLEX project, seismic CPT have been carried out at sites where in-situ undisturbed samples of sand have been obtained using in-situ ground freezing (Sego et al., 1994). Based on a comparison between predicted void ratio and measured void ratio, the influence of aging on normalized shear wave velocity can be quantified. Figure 2 shows a plot of the change in normalized shear wave velocity versus age for the CANLEX sites. Although limited data are available for the influence of aging on CPT penetration resistance, there is evidence of aging on the SPT penetration resistance (Skempton, 1986; Kulhawy and Mayne, 1990). Therefore, both shear wave velocity and penetration resistance increase with age for a given sand. However, due to the nature of the proposed correlation given in equation 3, the parameter Y will increase with age.

Based on a number of sites where seismic CPT has been carried out as well as laboratory testing to determine the slope of the steady state line, a preliminary correlation between the slope of the steady state line ( $\lambda$ ) and the parameter  $Y$  has been developed and is shown on Figure 3. Included on Figure 3 are estimated contours of age. Figure 3 applies only to uncemented sand.

Gillespie (1988) suggested a chart to estimate soil behavior type based on CPT penetration resistance ( $q_c$ ) and the ratio of small strain shear modulus to cone penetration resistance, ( $G_0/q_c$ ). The small strain shear modulus,  $G_0$ , is linked to shear wave velocity as follows:

$$(4) \quad G_0 = \rho V_s^2$$

Gillespie (1988) found that the ratio  $G_0/q_c$  varied from about 4 to 20 in sands and increased in silty sands, silts and clays. Mayne and Rix (1993) showed that the ratio  $G_0/q_c$  varied with void ratio and cone penetration resistance for a wide range of clays. A modified and updated version of the soil classification chart suggested by Gillespie is shown in Figure 4. The ratio  $G_0/q_c$  is similar to the parameter  $Y$ , although the exponent is slightly different. Since increasing sand compressibility will decrease the cone penetration resistance and increasing age and cementation will tend to increase the penetration resistance, it is possible to estimate the regions on Figure 4 where aged, cemented and compressible sands should fall. These have been included on Figure 4 as a guide. Currently there is limited data available to fully evaluate the chart for aged and cemented soils. This chart can be used to estimate soil type. The chart is especially useful for identifying 'unusual' soils such as cemented or aged soils that fall outside of the central region of the chart. The chart can also be used to identify structured clays and fine grained soils that have either a high or low water content.

## SUMMARY

Interpretation of CPT results is primarily done using empirical correlations. These empirical correlations have generally been developed based on extensive CPT data in certain soil types. The correlations can be expected to provide reasonable results provided the soil is similar to those in which the correlation was based. The interpretation of parameters such as relative density or state parameter from CPT results is generally restricted to young, unaged, moderately compressible silica sands. Interpretation of CPT results in sands that have a high compressibility, such as sands with a high fines content or with a high mica or carbonate content can be difficult and can produce misleading results. Therefore, there is a need to have the ability to identify 'unusual' soil types from CPT results so that modifications can be made to the empirical correlations to reflect the soil type. For the interpretation of CPT results in sandy soils it is important to be able to identify highly compressible sands. However, this is difficult based only on the basic CPT data. Hence, a preliminary method has been proposed based on the combined results of penetration resistance and shear wave velocity from the seismic CPT. A new soil behavior type chart has been developed based on normalized cone penetration resistance and the ratio of small strain shear modulus with cone penetration resistance. The small strain shear modulus is derived directly from the measured shear wave velocity. This chart provides some guide for the identification of 'unusual' soils, such as, either compressible or cemented or aged sands.

## ACKNOWLEDGMENTS

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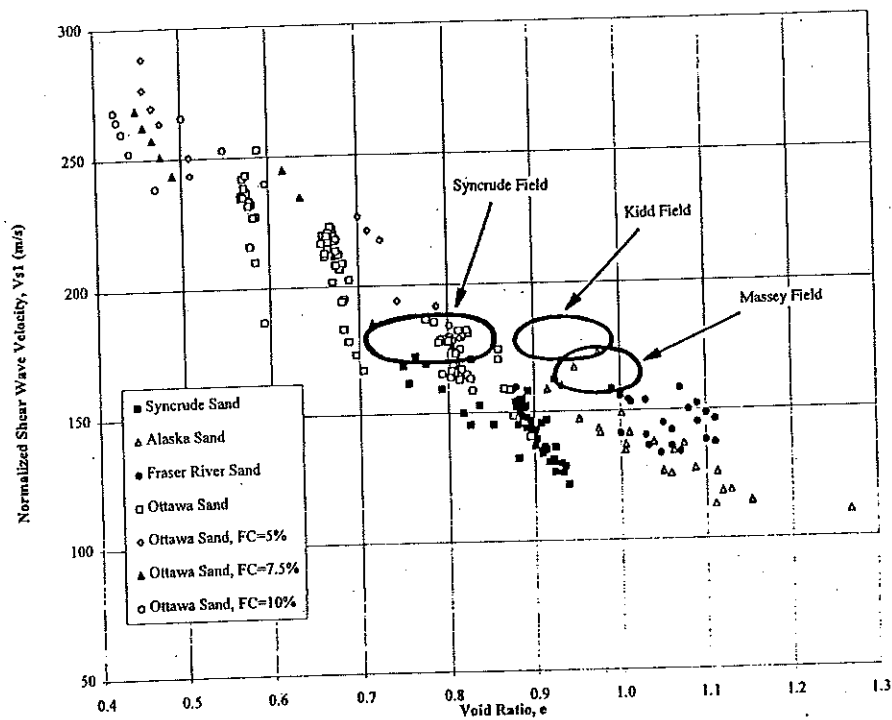


Figure 1. Normalized shear wave velocity versus void ratio for a wide range of sands.

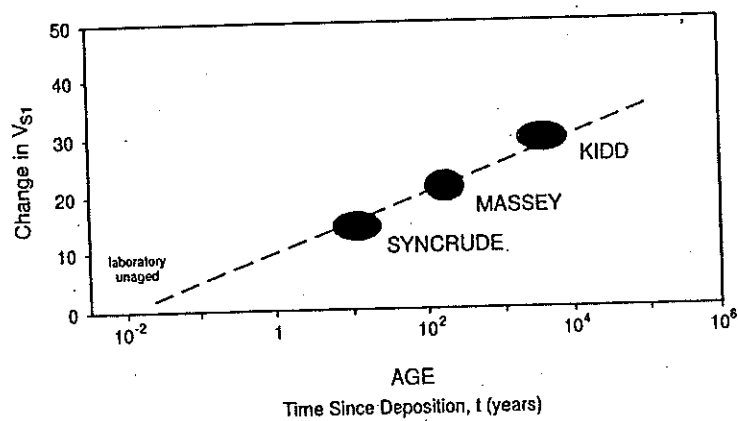


Figure 2. Proposed change in normalized shear wave ( $V_{s1}$ ) velocity in sands due to aging.

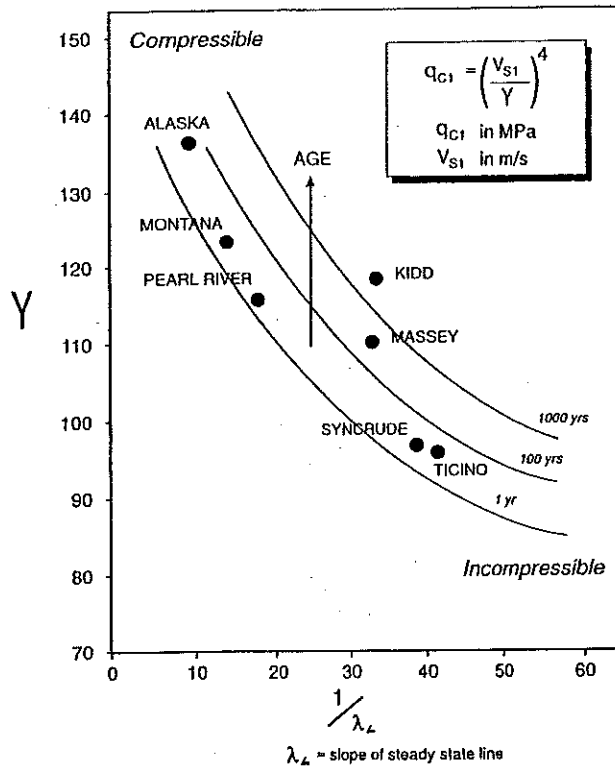


Figure 4. Proposed correlation for estimating slope of the steady state line ( $\lambda$ ) for uncemented sand from seismic CPT results.

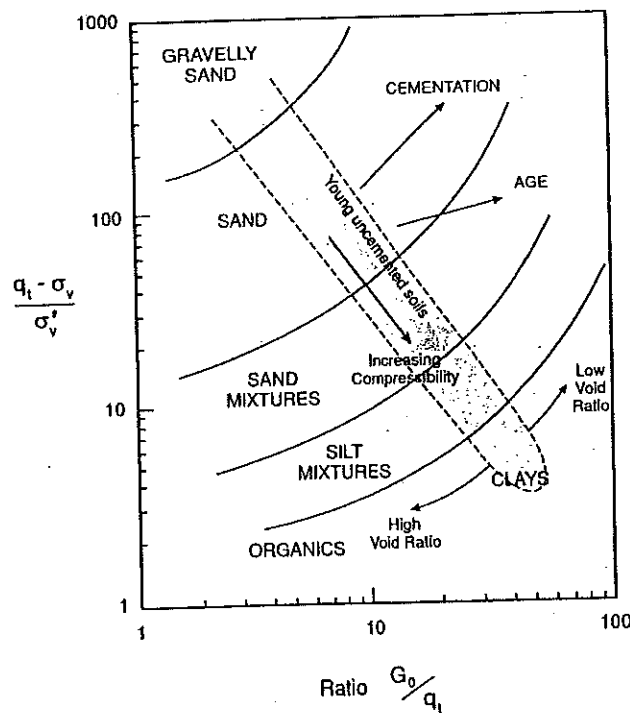
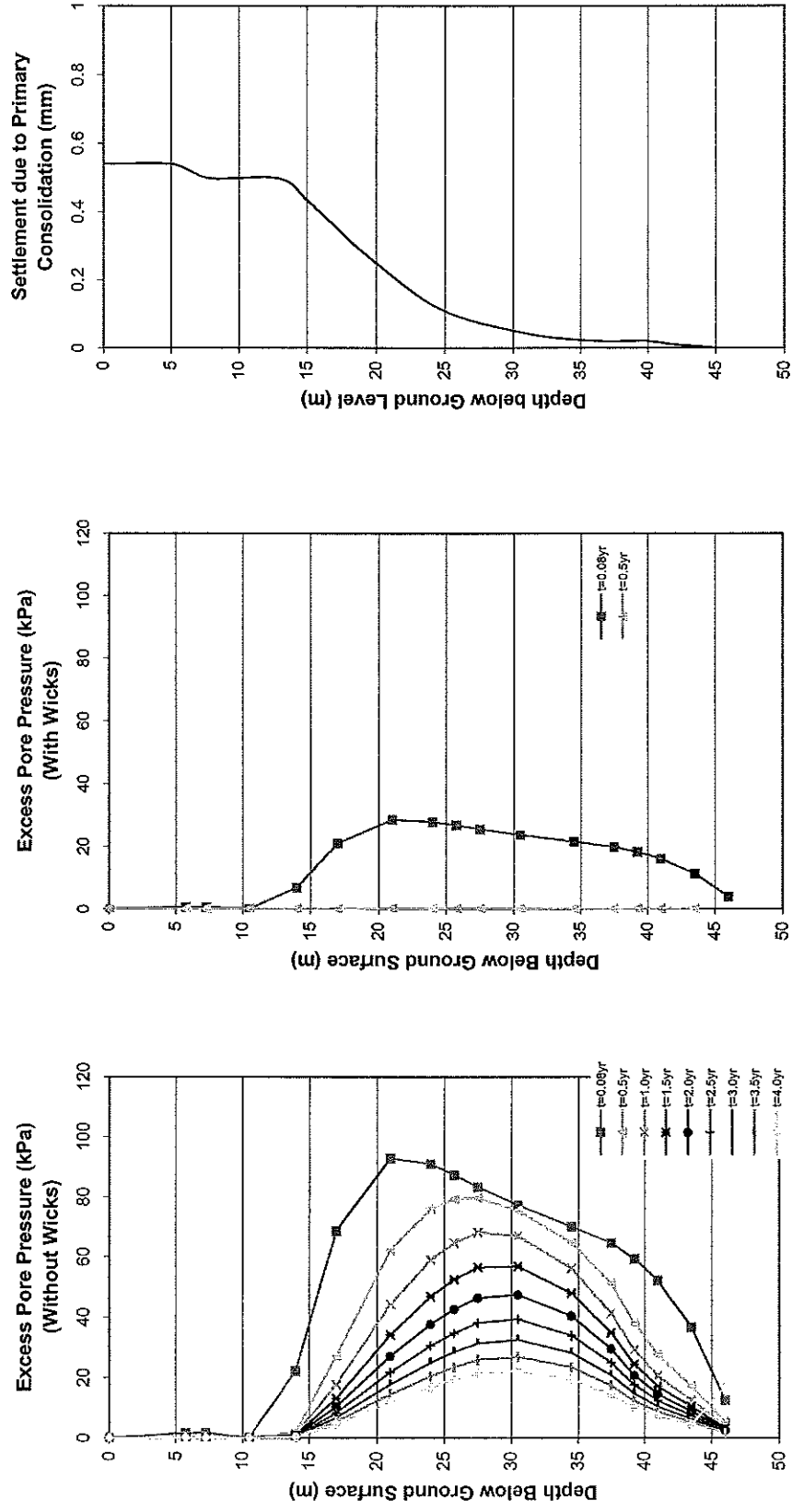


Figure 5. Proposed soil behavior type chart based on normalized CPT penetration resistance and the ratio of small strain shear modulus with penetration resistance ( $G_0/q_c$ ).

**Appendix F**  
**Primary Consolidation Analysis Results**

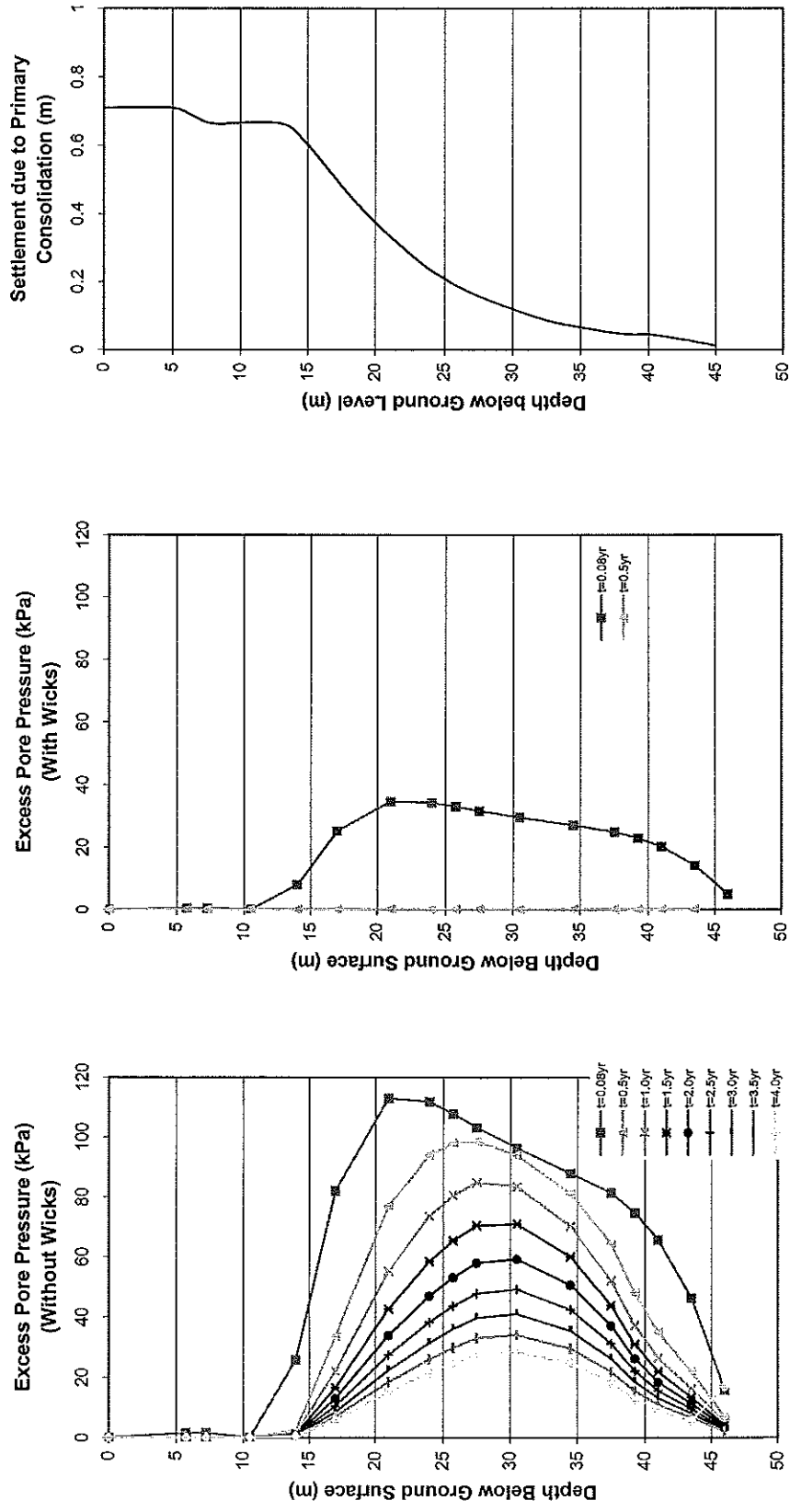
HIGHWAY 69 - FOUR LANE - CNR CROSSING NORTH OF ESTAIRE  
SETTLEMENT AND EXCESS PORE PRESSURE PLOTS  
SOUTHBOUND LANE BRIDGE (SBL) - North Abutment (10+550 to 10+600)

Case A: 6m Rockfill Embankment + 3m Surcharge (O/C)



HIGHWAY 69 - FOUR LANE - CNR CROSSING NORTH OF ESTAIRE  
SETTLEMENT AND EXCESS PORE PRESSURE PLOTS  
SOUTHBOUND LANE BRIDGE (SBL) - North Abutment (10+550 to 10+600)

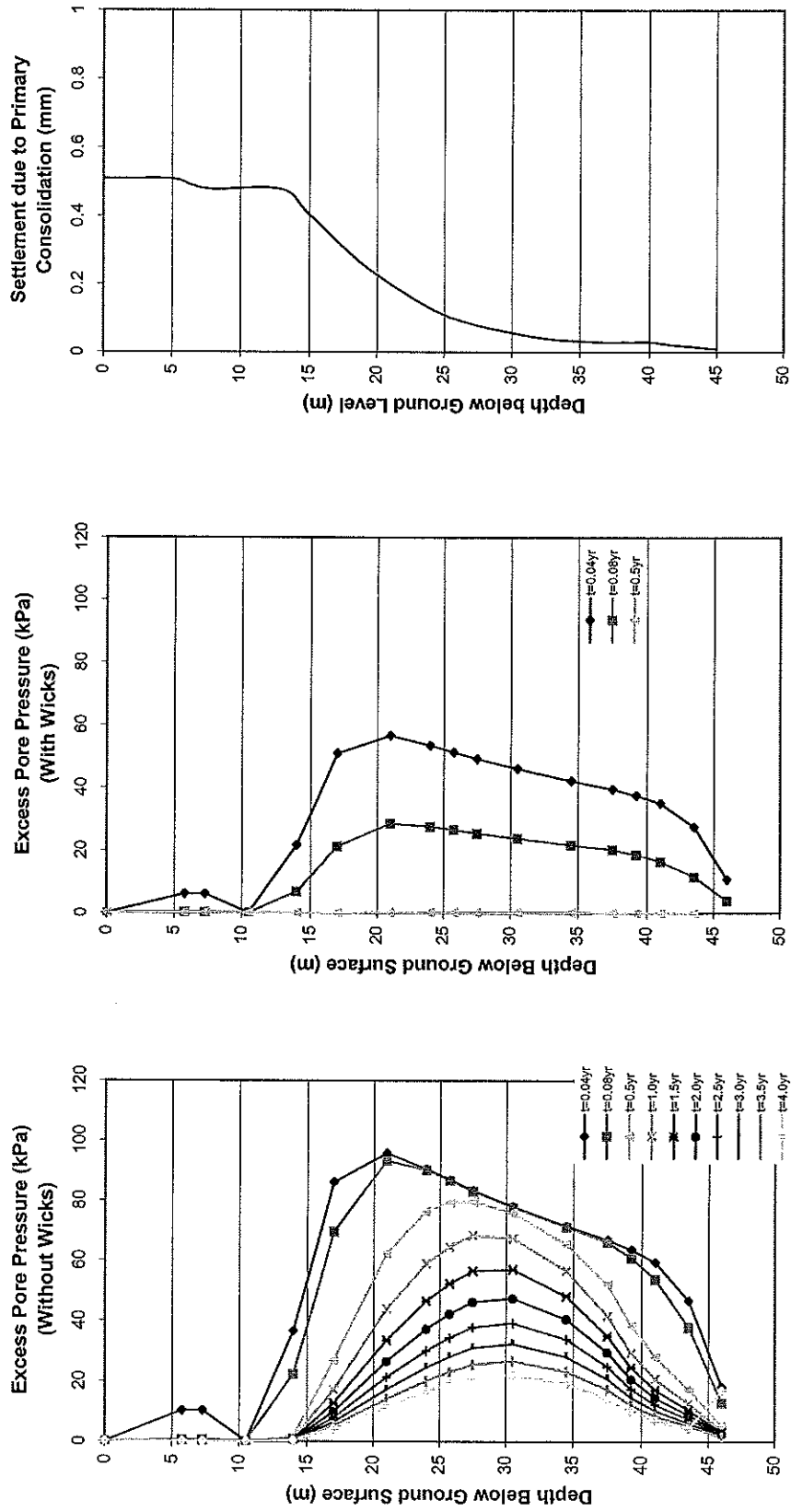
Case C: 6m Sandfill Embankment + 3m Surcharge (O/C)





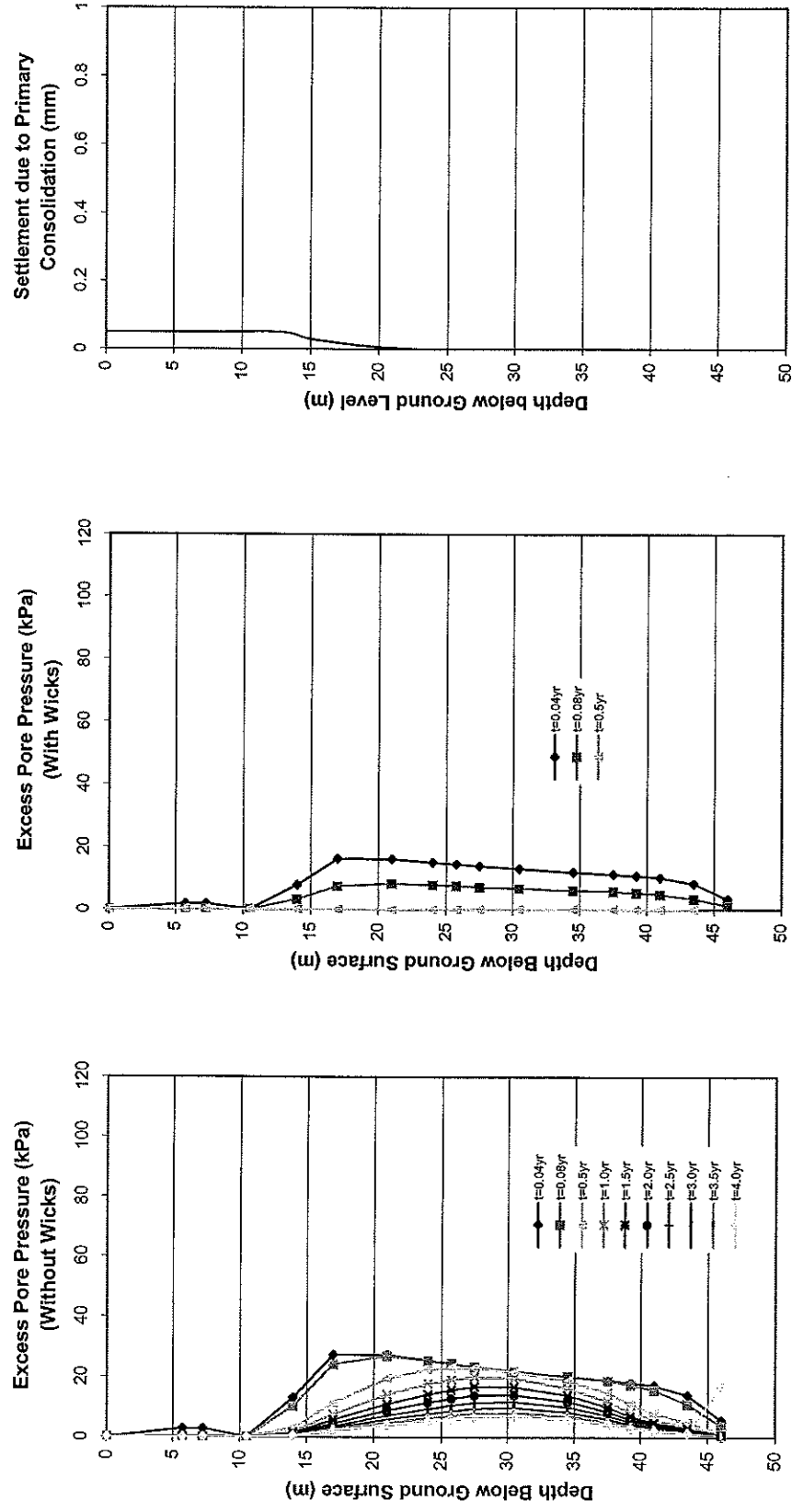
HIGHWAY 69 - FOUR LANING - CNR CROSSING NORTH OF ESTAIRE  
SETTLEMENT AND EXCESS PORE PRESSURE PLOTS  
SOUTHBOUND LANE BRIDGE (SBL) - North Abutment (10+550 to 10+600)

Case D: 6m Sandfill Embankment (O/C)



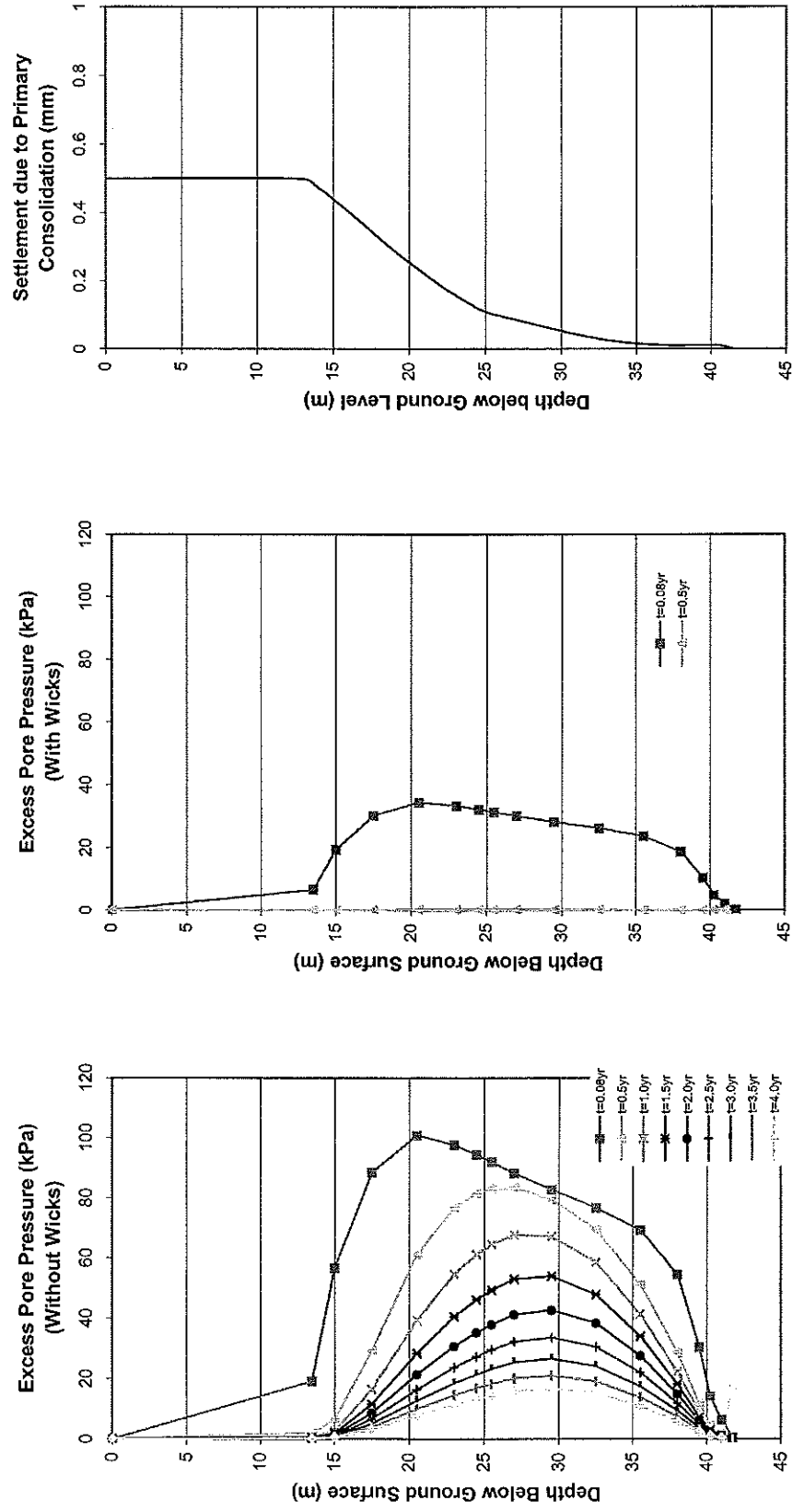
**HIGHWAY 69 - FOUR LANE - CNR CROSSING NORTH OF ESTAIRE**  
**SETTLEMENT AND EXCESS PORE PRESSURE PLOTS**  
**SOUTHBOUND LANE BRIDGE (SBL) - North Abutment (10+550 to 10+600)**

**Case E: 6m REP Embankment and Sandfill Cover(O/C)**



**HIGHWAY 69 - FOUR LANE - CNR CROSSING NORTH OF ESTAIRE  
SETTLEMENT AND EXCESS PORE PRESSURE PLOTS  
NORTHBOUND LANE BRIDGE (NBL) - North Abutment (10+580 to 10+620)**

**Case A: 6.3m Rockfill Embankment + 3m Surcharge (O/C)**



HIGHWAY 69 - FOUR LANING - CNR CROSSING NORTH OF ESTAIRE  
SETTLEMENT AND EXCESS PORE PRESSURE PLOTS  
NORTHBOUND LANE BRIDGE (NBL) - North Abutment (10+580 to 10+620)

Case C: 6.3m Sandfill Embankment + 3m Surcharge (O/C)

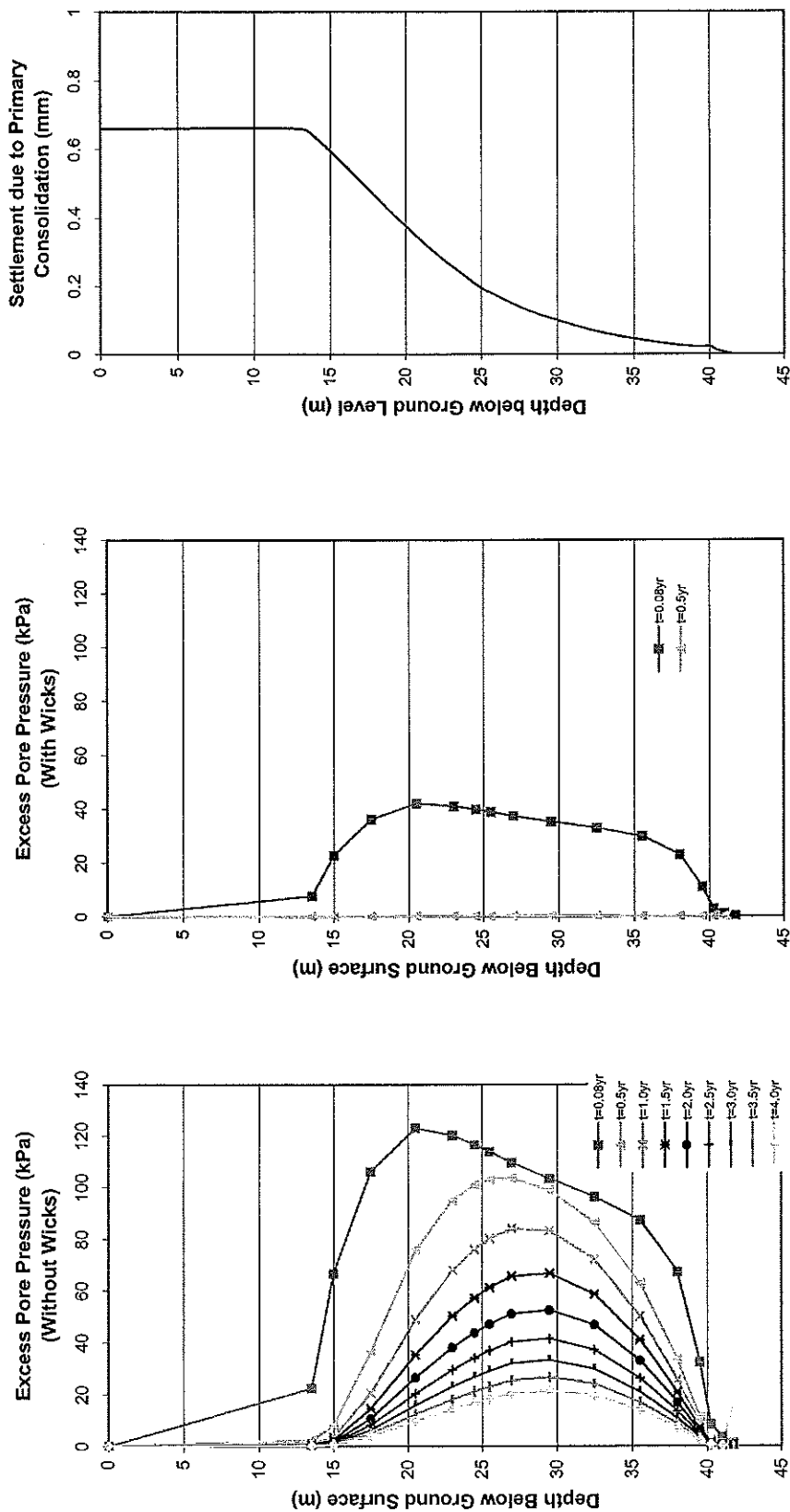
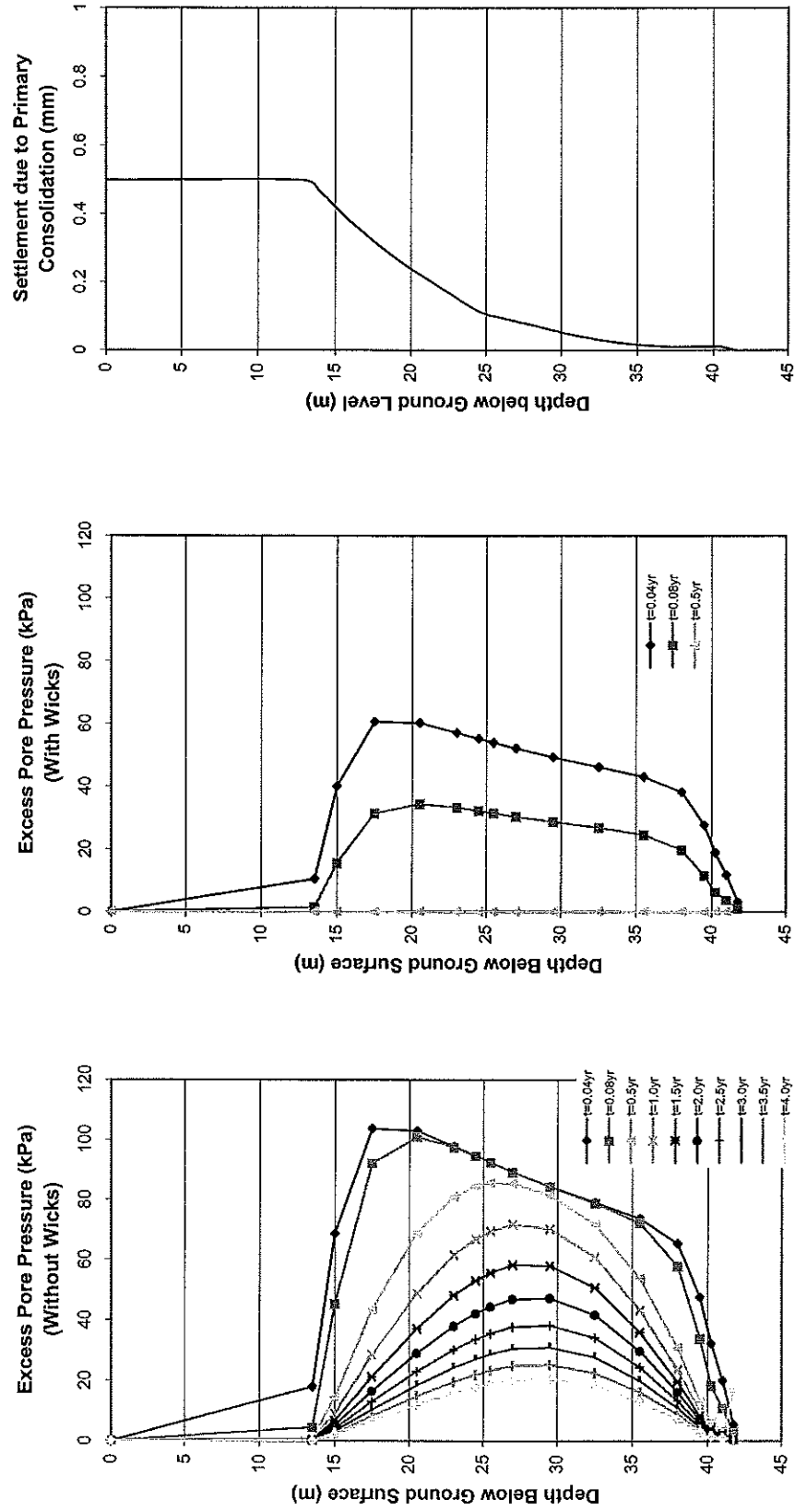


FIGURE F6

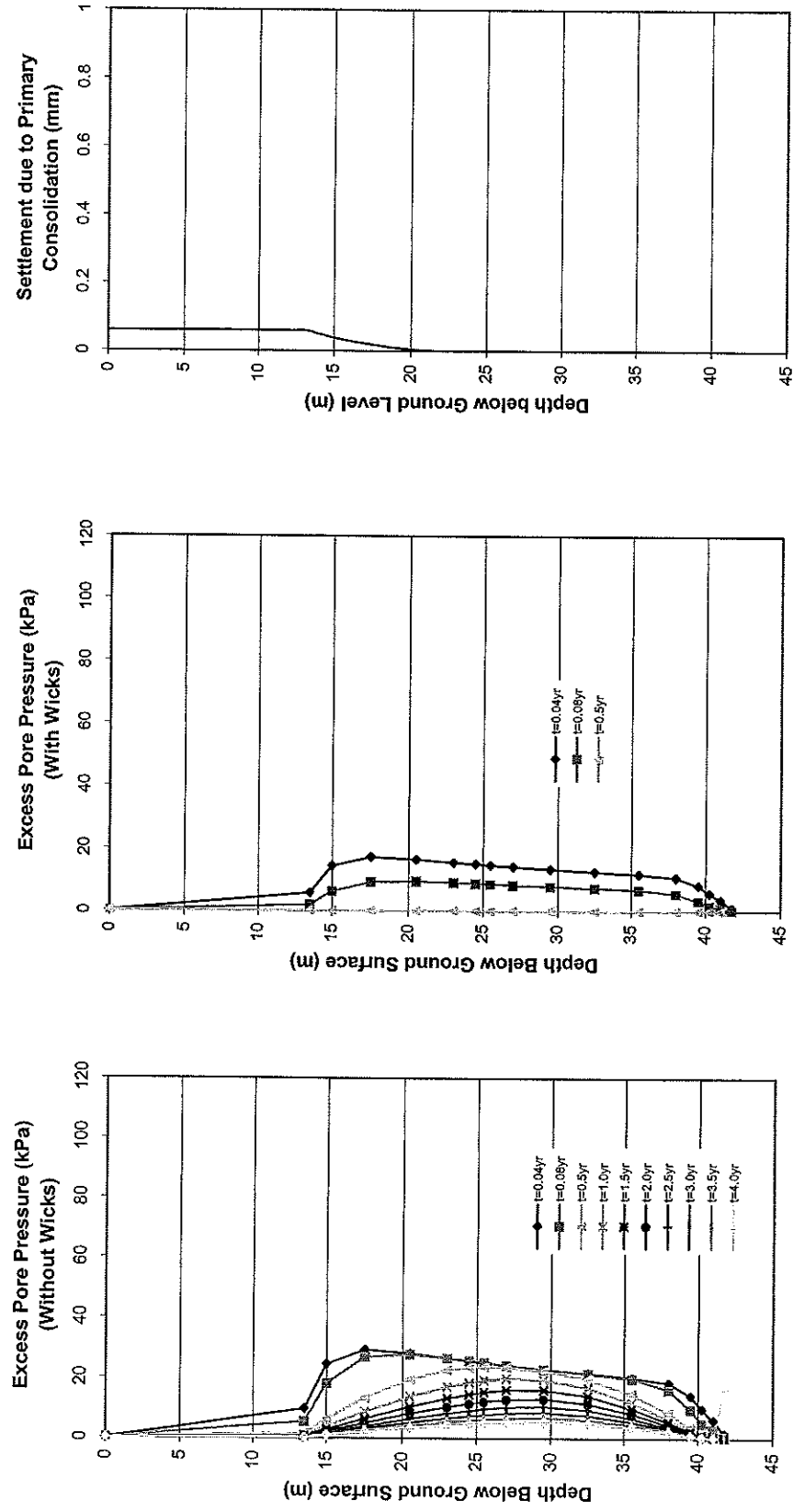
NBLN-AbutC

HIGHWAY 69 - FOUR LANE - CNR CROSSING NORTH OF ESTAIRE  
SETTLEMENT AND EXCESS PORE PRESSURE PLOTS  
NORTHBOUND LANE BRIDGE (NBL) - North Abutment (10+580 to 10+620)

Case D: 6.3m Sandfill Embankment (O/C)



**HIGHWAY 69 - FOUR LANE - CNR CROSSING NORTH OF ESTAIRE**  
**SETTLEMENT AND EXCESS PORE PRESSURE PLOTS**  
**NORTHBOUND LANE BRIDGE (NBL) - North Abutment (10+580 to 10+620)**  
**Case E: 6.3m REP Embankment and Sandfill Cover(O/C)**

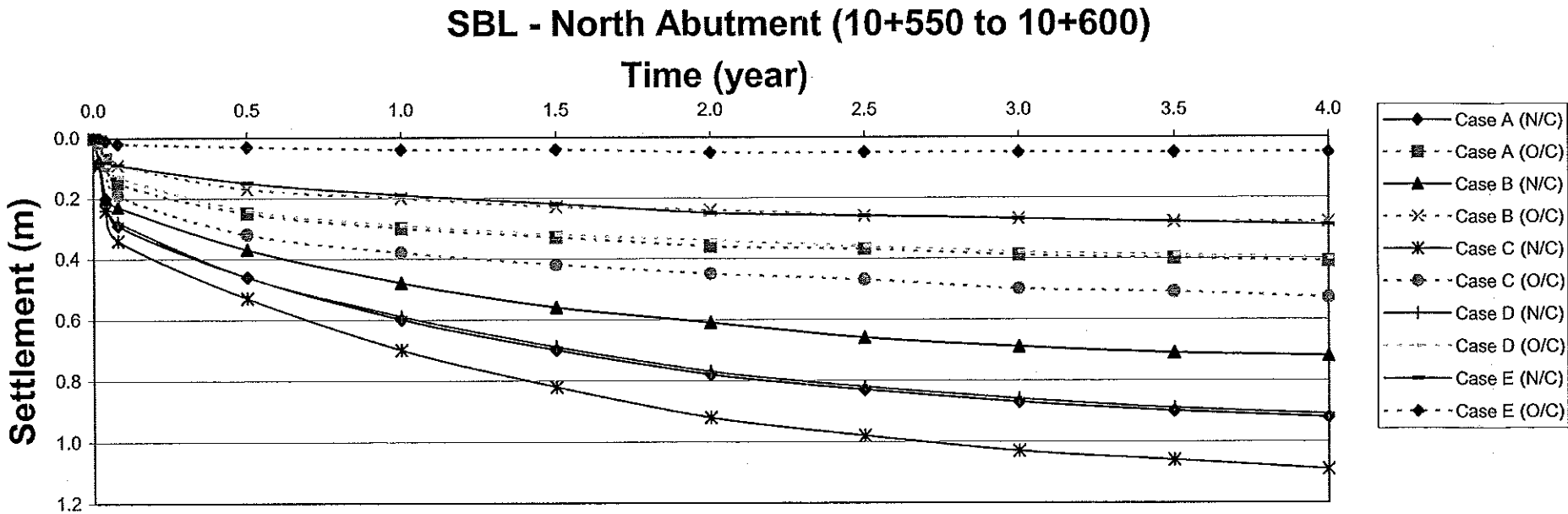


HIGHWAY 69 - FOUR LANING - CNR CROSSING NORTH OF ESTAIRE  
SUMMARY OF SETTLEMENT DUE TO PRIMARY CONSOLIDATION  
SOUTHBOUND LANE BRIDGE (SBL) - North Abutment (10+550 to 10+600)

- Case A: 6m Rockfill Embankment + 3m Surcharge  
Case B: 6m Rockfill Embankment  
Case C: 6m Sandfill Embankment + 3m Surcharge  
Case D: 6m Sandfill Embankment  
Case E: 6m REP Embankment and Sandfill Cover

Note
1. N/C: Normal Consolidated
2. O/C: Over Consolidated

Time from Beginning of Construction (yr)	SBL - North Abutment (10+550 to 10+600)																			
	Settlement (m) and % of Consolidation at centerline of SBL																			
	Case A				Case B				Case C				Case D				Case E			
	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%
0.00	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%
0.02	0.08	8%	0.01	2%	0.08	10%	0.01	3%	0.09	7%	0.02	3%	0.09	9%	0.02	4%	0.01	3%	0.00	0%
0.04	0.20	19%	0.07	13%	0.20	24%	0.07	21%	0.24	19%	0.09	13%	0.24	23%	0.09	17%	0.08	24%	0.01	20%
0.08	0.29	28%	0.15	28%	0.23	28%	0.09	26%	0.34	27%	0.19	27%	0.28	27%	0.13	25%	0.09	26%	0.02	40%
0.50	0.46	44%	0.25	46%	0.37	45%	0.17	50%	0.53	43%	0.32	45%	0.46	45%	0.24	46%	0.15	44%	0.03	60%
1.00	0.60	57%	0.30	56%	0.48	58%	0.20	59%	0.70	56%	0.38	54%	0.59	57%	0.29	56%	0.19	56%	0.04	80%
1.50	0.70	67%	0.33	61%	0.56	67%	0.23	68%	0.82	66%	0.42	59%	0.69	67%	0.32	62%	0.22	65%	0.04	80%
2.00	0.78	74%	0.36	67%	0.61	73%	0.24	71%	0.92	74%	0.45	63%	0.77	75%	0.34	65%	0.25	74%	0.05	100%
2.50	0.83	79%	0.37	69%	0.66	80%	0.26	76%	0.98	79%	0.47	66%	0.82	80%	0.36	69%	0.26	76%	0.05	100%
3.00	0.87	83%	0.39	72%	0.69	83%	0.27	79%	1.03	83%	0.50	70%	0.86	83%	0.38	73%	0.27	79%	0.05	100%
3.50	0.90	86%	0.40	74%	0.71	86%	0.28	82%	1.06	85%	0.51	72%	0.89	86%	0.39	75%	0.28	82%	0.05	100%
4.00	0.92	88%	0.41	76%	0.72	87%	0.28	82%	1.09	88%	0.53	75%	0.91	88%	0.40	77%	0.29	85%	0.05	100%
Ultimate Settlement due to Primary Consolidation	1.05	100%	0.54	100%	0.83	100%	0.34	100%	1.24	100%	0.71	100%	1.03	100%	0.52	100%	0.34	100%	0.05	100%



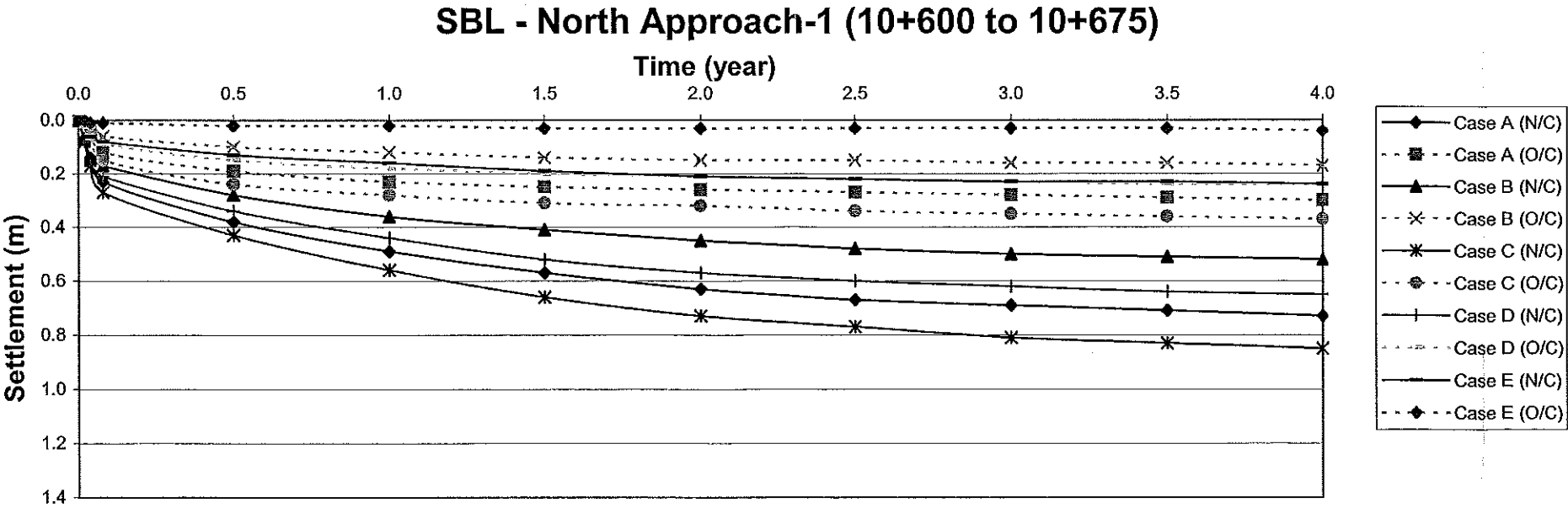
HIGHWAY 69 - FOUR LANING - CNR CROSSING NORTH OF ESTAIRE  
SUMMARY OF SETTLEMENT DUE TO PRIMARY CONSOLIDATION  
SOUTHBOUND LANE BRIDGE (SBL) - North Approach-1 (10+600 to 10+675)

- Case A: 4.5m Rockfill Embankment + 3m Surcharge  
Case B: 4.5m Rockfill Embankment  
Case C: 4.5m Sandfill Embankment + 3m Surcharge  
Case D: 4.5m Sandfill Embankment  
Case E: 4.5m REP Embankment and Sandfill Cover

Note

1. N/C: Normal Consolidated  
2. O/C: Over Consolidated

Time from Beginning of Construction (yr)	SBL - North Approach-1 (10+600 to 10+675)																			
	Settlement (m) and % of Consolidation at centerline of SBL																			
	Case A				Case B				Case C				Case D				Case E			
	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%
0.00	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%
0.02	0.07	9%	0.01	3%	0.07	12%	0.01	5%	0.08	8%	0.01	2%	0.08	11%	0.01	3%	0.01	4%	0.00	0%
0.04	0.14	17%	0.04	11%	0.14	23%	0.04	20%	0.17	18%	0.06	13%	0.17	23%	0.06	21%	0.06	21%	0.01	25%
0.08	0.23	28%	0.12	33%	0.17	28%	0.06	30%	0.27	28%	0.15	32%	0.21	28%	0.09	31%	0.08	29%	0.01	25%
0.50	0.38	46%	0.19	53%	0.28	47%	0.10	50%	0.43	45%	0.24	51%	0.34	46%	0.15	52%	0.13	46%	0.02	50%
1.00	0.49	60%	0.23	64%	0.36	60%	0.12	60%	0.56	58%	0.28	60%	0.44	59%	0.18	62%	0.16	57%	0.02	50%
1.50	0.57	70%	0.25	69%	0.41	68%	0.14	70%	0.66	69%	0.31	66%	0.52	70%	0.20	69%	0.19	68%	0.03	75%
2.00	0.63	77%	0.26	72%	0.45	75%	0.15	75%	0.73	76%	0.32	68%	0.57	77%	0.21	72%	0.21	75%	0.03	75%
2.50	0.67	82%	0.27	75%	0.48	80%	0.15	75%	0.77	80%	0.34	72%	0.60	81%	0.22	76%	0.22	79%	0.03	75%
3.00	0.69	84%	0.28	78%	0.50	83%	0.16	80%	0.81	84%	0.35	74%	0.62	84%	0.23	79%	0.23	82%	0.03	75%
3.50	0.71	87%	0.29	81%	0.51	85%	0.16	80%	0.83	86%	0.36	77%	0.64	86%	0.24	83%	0.23	82%	0.03	75%
4.00	0.73	89%	0.30	83%	0.52	87%	0.17	85%	0.85	89%	0.37	79%	0.65	88%	0.24	83%	0.24	86%	0.04	100%
Ultimate Settlement due to Primary Consolidation	0.82	100%	0.36	100%	0.60	100%	0.20	100%	0.96	100%	0.47	100%	0.74	100%	0.29	100%	0.28	100%	0.04	100%





HIGHWAY 69 - FOUR LANING - CNR CROSSING NORTH OF ESTAIRE  
SUMMARY OF SETTLEMENT DUE TO PRIMARY CONSOLIDATION  
SOUTHBOUND LANE BRIDGE (SBL) - North Approach-2 (10+675 to Dill TWP 10+010)

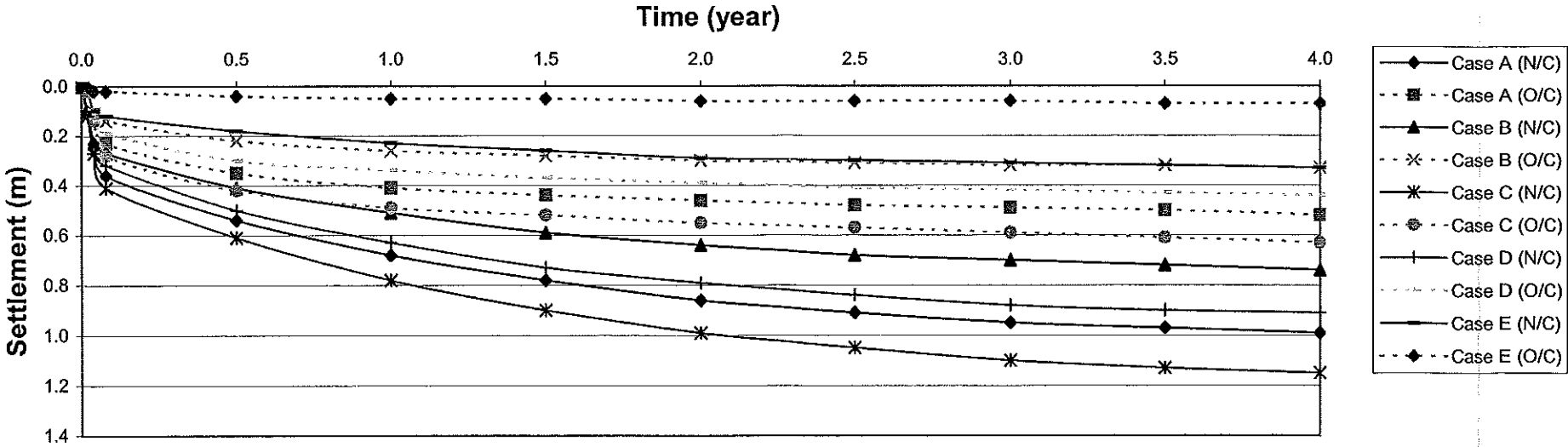
- Case A: 5m Rockfill Embankment + 3m Surcharge  
Case B: 5m Rockfill Embankment  
Case C: 5m Sandfill Embankment + 3m Surcharge  
Case D: 5m Sandfill Embankment  
Case E: 5m REP Embankment and Sandfill Cover

Note

1. N/C: Normal Consolidated  
2. O/C: Over Consolidated

Time from Beginning of Construction (yr)	SBL - North Approach-2 (10+675 to Dill TWP 10+010)																			
	Settlement (m) and % of Consolidation at centerline of SBL																			
	Case A				Case B				Case C				Case D				Case E			
	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%
0.00	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%
0.02	0.10	9%	0.02	3%	0.10	12%	0.02	5%	0.12	9%	0.02	3%	0.12	12%	0.02	4%	0.01	3%	0.00	0%
0.04	0.23	21%	0.11	17%	0.23	27%	0.11	28%	0.27	21%	0.14	18%	0.27	26%	0.14	26%	0.10	26%	0.02	22%
0.08	0.36	32%	0.23	37%	0.27	32%	0.14	36%	0.41	32%	0.28	35%	0.32	31%	0.19	35%	0.12	32%	0.02	22%
0.50	0.54	48%	0.35	56%	0.41	49%	0.22	56%	0.61	47%	0.42	53%	0.50	48%	0.30	56%	0.18	47%	0.04	44%
1.00	0.68	61%	0.41	65%	0.51	61%	0.26	67%	0.78	60%	0.49	62%	0.63	61%	0.34	63%	0.23	61%	0.05	56%
1.50	0.78	70%	0.44	70%	0.59	70%	0.28	72%	0.90	69%	0.52	66%	0.73	70%	0.37	69%	0.26	68%	0.05	56%
2.00	0.86	77%	0.46	73%	0.64	76%	0.30	77%	0.99	76%	0.55	70%	0.79	76%	0.39	72%	0.29	76%	0.06	67%
2.50	0.91	81%	0.48	76%	0.68	81%	0.31	79%	1.05	81%	0.57	72%	0.84	81%	0.41	76%	0.30	79%	0.06	67%
3.00	0.95	85%	0.49	78%	0.70	83%	0.32	82%	1.10	85%	0.59	75%	0.88	85%	0.42	78%	0.31	82%	0.06	67%
3.50	0.97	87%	0.50	79%	0.72	86%	0.32	82%	1.13	87%	0.61	77%	0.90	87%	0.43	80%	0.32	84%	0.07	78%
4.00	0.99	88%	0.52	83%	0.74	88%	0.33	85%	1.15	88%	0.63	80%	0.91	88%	0.44	81%	0.33	87%	0.07	78%
Ultimate Settlement due to Primary Consolidation	1.12	100%	0.63	100%	0.84	100%	0.39	100%	1.30	100%	0.79	100%	1.04	100%	0.54	100%	0.38	100%	0.09	100%

SBL - North Approach-2 (10+675 to Dill TWP 10+010)



HIGHWAY 69 - FOUR LANING - CNR CROSSING NORTH OF ESTAIRE  
SUMMARY OF SETTLEMENT DUE TO PRIMARY CONSOLIDATION  
SOUTHBOUND LANE BRIDGE (SBL) - North Approach-3 (Dill TWP10+010 to 10+040)

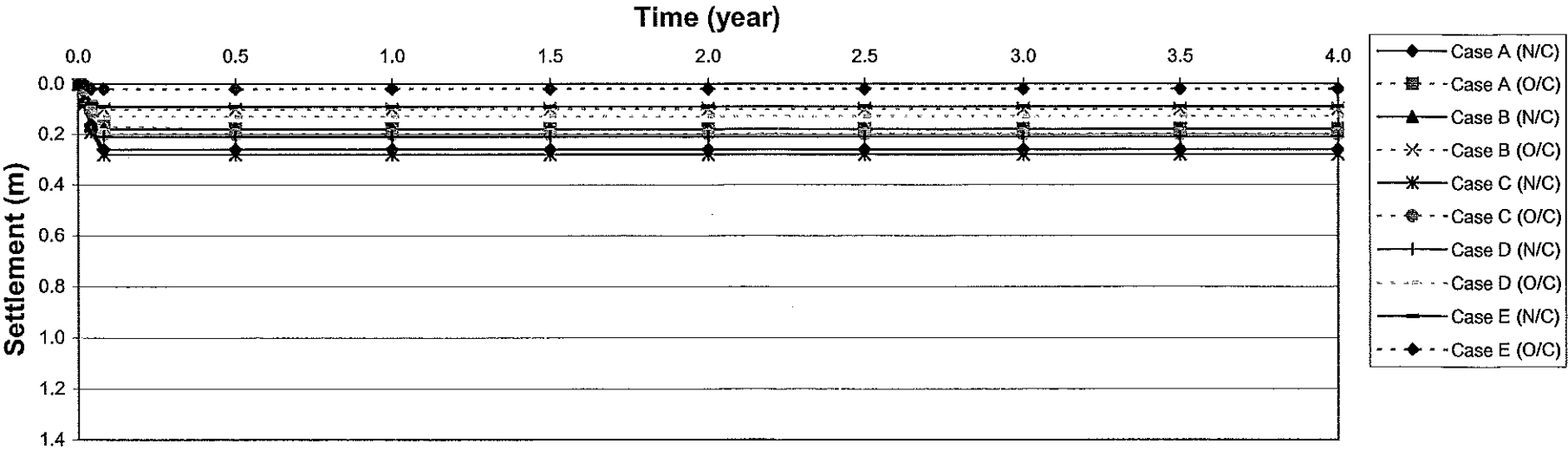
- Case A: 4.2m Rockfill Embankment + 3m Surcharge
- Case B: 4.2m Rockfill Embankment
- Case C: 4.2m Sandfill Embankment + 3m Surcharge
- Case D: 4.2m Sandfill Embankment
- Case E: 4.2m REP Embankment and Sandfill Cover

Note

1. N/C: Normal Consolidated
2. O/C: Over Consolidated

Time from Beginning of Construction (yr)	SBL - North Approach-3 (Dill TWP10+010 to 10+040)																			
	Settlement (m) and % of Consolidation at centerline of SBL																			
	Case A				Case B				Case C				Case D				Case E			
	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%
0.00	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%
0.02	0.07	27%	0.02	11%	0.07	39%	0.02	20%	0.08	29%	0.02	10%	0.08	38%	0.02	15%	0.01	11%	0.00	0%
0.04	0.16	62%	0.09	50%	0.16	89%	0.09	90%	0.19	68%	0.11	55%	0.19	90%	0.11	85%	0.08	89%	0.02	100%
0.08	0.26	100%	0.17	94%	0.18	100%	0.10	100%	0.28	100%	0.20	100%	0.21	100%	0.13	100%	0.09	100%	0.02	100%
0.50	0.26	100%	0.18	100%	0.18	100%	0.10	100%	0.28	100%	0.20	100%	0.21	100%	0.13	100%	0.09	100%	0.02	100%
1.00	0.26	100%	0.18	100%	0.18	100%	0.10	100%	0.28	100%	0.20	100%	0.21	100%	0.13	100%	0.09	100%	0.02	100%
1.50	0.26	100%	0.18	100%	0.18	100%	0.10	100%	0.28	100%	0.20	100%	0.21	100%	0.13	100%	0.09	100%	0.02	100%
2.00	0.26	100%	0.18	100%	0.18	100%	0.10	100%	0.28	100%	0.20	100%	0.21	100%	0.13	100%	0.09	100%	0.02	100%
2.50	0.26	100%	0.18	100%	0.18	100%	0.10	100%	0.28	100%	0.20	100%	0.21	100%	0.13	100%	0.09	100%	0.02	100%
3.00	0.26	100%	0.18	100%	0.18	100%	0.10	100%	0.28	100%	0.20	100%	0.21	100%	0.13	100%	0.09	100%	0.02	100%
3.50	0.26	100%	0.18	100%	0.18	100%	0.10	100%	0.28	100%	0.20	100%	0.21	100%	0.13	100%	0.09	100%	0.02	100%
4.00	0.26	100%	0.18	100%	0.18	100%	0.10	100%	0.28	100%	0.20	100%	0.21	100%	0.13	100%	0.09	100%	0.02	100%
Ultimate Settlement due to Primary Consolidation	0.26	100%	0.18	100%	0.18	100%	0.10	100%	0.28	100%	0.20	100%	0.21	100%	0.13	100%	0.09	100%	0.02	100%

SBL - North Approach-3 (Dill TWP10+010 to 10+040)



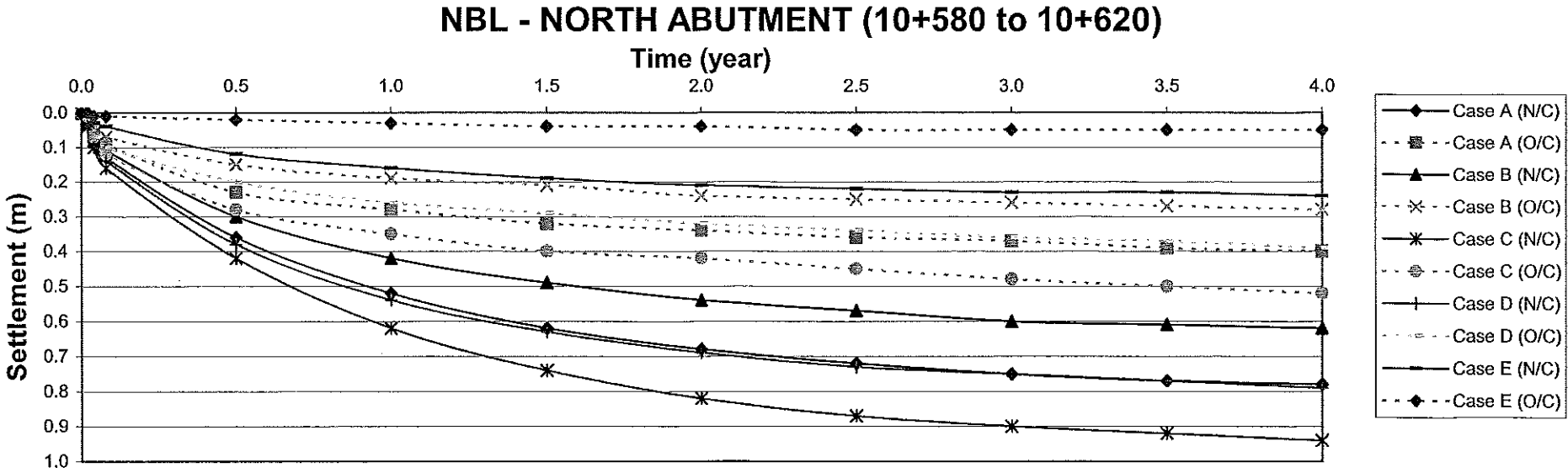
HIGHWAY 69 - FOUR LANING - CNR CROSSING NORTH OF ESTAIRE  
SUMMARY OF SETTLEMENT DUE TO PRIMARY CONSOLIDATION  
NORTHBOUND LANE BRIDGE (NBL) - NORTH ABUTMENT (10+580 to 10+620)

Case A: 6.3m Rockfill Embankment + 3m Surcharge  
Case B: 6.3m Rockfill Embankment  
Case C: 6.3m Sandfill Embankment + 3m Surcharge  
Case D: 6.3m Sandfill Embankment  
Case E: 6.3m REP Embankment and Sandfill Cover

Note

1. N/C: Normal Consolidated
2. O/C: Over Consolidated

Time from Beginning of Construction (yr)	NBL - NORTH ABUTMENT (10+580 to 10+620)																			
	Settlement (m) and % of Consolidation at centerline of NBL																			
	Case A				Case B				Case C				Case D				Case E			
	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%
0.00	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%
0.02	0.03	3%	0.01	2%	0.03	4%	0.01	3%	0.04	4%	0.01	2%	0.04	5%	0.01	2%	0.00	0%	0.00	0%
0.04	0.08	9%	0.05	10%	0.08	11%	0.05	14%	0.10	10%	0.07	11%	0.10	11%	0.07	14%	0.03	11%	0.01	14%
0.08	0.13	15%	0.09	18%	0.11	16%	0.07	20%	0.16	15%	0.12	18%	0.14	16%	0.10	20%	0.04	15%	0.01	14%
0.50	0.36	41%	0.23	46%	0.30	43%	0.15	43%	0.42	40%	0.28	42%	0.38	43%	0.20	40%	0.12	44%	0.02	29%
1.00	0.52	59%	0.28	56%	0.42	60%	0.19	54%	0.62	59%	0.35	53%	0.54	61%	0.26	52%	0.16	59%	0.03	43%
1.50	0.62	70%	0.32	64%	0.49	70%	0.21	60%	0.74	70%	0.40	61%	0.63	72%	0.29	58%	0.19	70%	0.04	57%
2.00	0.68	77%	0.34	68%	0.54	77%	0.24	69%	0.82	78%	0.42	64%	0.69	78%	0.32	64%	0.21	78%	0.04	57%
2.50	0.72	82%	0.36	72%	0.57	81%	0.25	71%	0.87	83%	0.45	68%	0.73	83%	0.34	68%	0.22	81%	0.05	71%
3.00	0.75	85%	0.37	74%	0.60	86%	0.26	74%	0.90	86%	0.48	73%	0.75	85%	0.36	72%	0.23	85%	0.05	71%
3.50	0.77	88%	0.39	78%	0.61	87%	0.27	77%	0.92	88%	0.50	76%	0.77	88%	0.37	74%	0.23	85%	0.05	71%
4.00	0.78	89%	0.40	80%	0.62	89%	0.28	80%	0.94	90%	0.52	79%	0.79	90%	0.39	78%	0.24	89%	0.05	71%
Ultimate Settlement due to Primary Consolidation	0.88	100%	0.50	100%	0.70	100%	0.35	100%	1.05	100%	0.66	100%	0.88	100%	0.50	100%	0.27	100%	0.07	100%



HIGHWAY 69 - FOUR LANING - CNR CROSSING NORTH OF ESTAIRE  
SUMMARY OF SETTLEMENT DUE TO PRIMARY CONSOLIDATION  
NORTHBOUND LANE BRIDGE (NBL) - NORTH APPROACH-1 (10+620 to 10+675)

Case A: 4.5m Rockfill Embankment + 3m Surcharge  
Case B: 4.5m Rockfill Embankment  
Case C: 4.5m Sandfill Embankment + 3m Surcharge  
Case D: 4.5m Sandfill Embankment  
Case E: 4.5m REP Embankment and Sandfill Cover

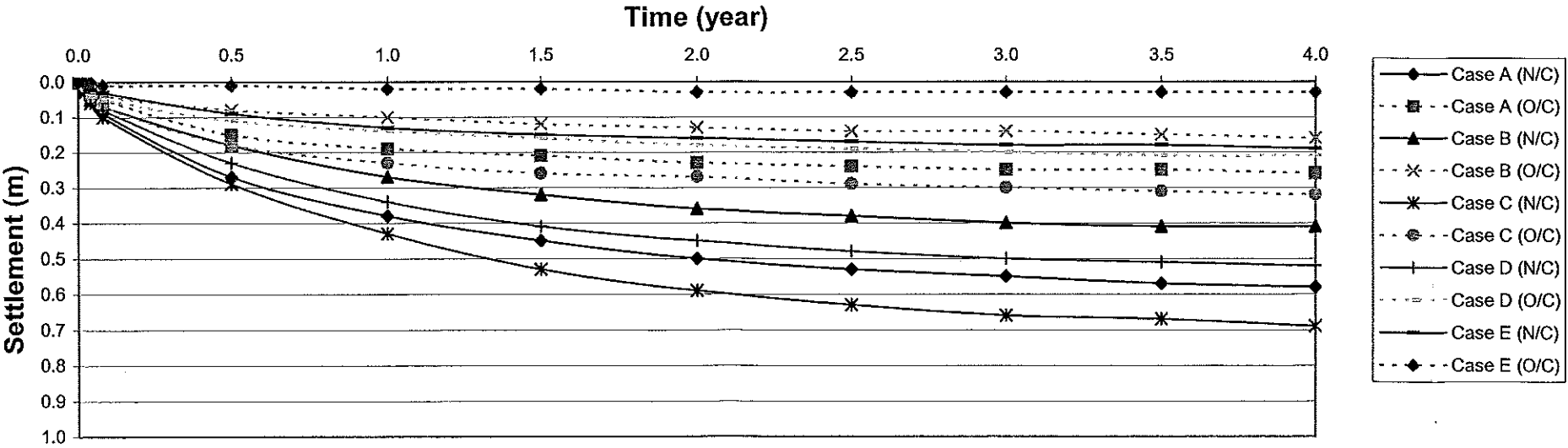
Note

1. N/C: Normal Consolidated

2. O/C: Over Consolidated

Time from Beginning of Construction (yr)	NBL - NORTH APPROACH-1 (10+620 to 10+675)																			
	Settlement (m) and % of Consolidation at centerline of NBL																			
	Case A				Case B				Case C				Case D				Case E			
	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%
0.00	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%
0.02	0.02	3%	0.00	0%	0.02	4%	0.00	0%	0.03	4%	0.01	3%	0.03	5%	0.01	4%	0.00	0%	0.00	0%
0.04	0.05	8%	0.03	11%	0.05	11%	0.03	17%	0.06	8%	0.04	11%	0.06	10%	0.04	16%	0.02	10%	0.00	0%
0.08	0.09	14%	0.05	18%	0.07	15%	0.04	22%	0.10	13%	0.07	19%	0.08	14%	0.05	20%	0.03	14%	0.01	25%
0.50	0.27	41%	0.15	54%	0.18	38%	0.08	44%	0.29	38%	0.18	50%	0.23	39%	0.11	44%	0.09	43%	0.01	25%
1.00	0.38	58%	0.19	68%	0.27	57%	0.10	56%	0.43	56%	0.23	64%	0.34	58%	0.14	56%	0.13	62%	0.02	50%
1.50	0.45	68%	0.21	75%	0.32	68%	0.12	67%	0.53	69%	0.26	72%	0.41	69%	0.16	64%	0.15	71%	0.02	50%
2.00	0.50	76%	0.23	82%	0.36	77%	0.13	72%	0.59	77%	0.27	75%	0.45	76%	0.18	72%	0.16	76%	0.03	75%
2.50	0.53	80%	0.24	86%	0.38	81%	0.14	78%	0.63	82%	0.29	81%	0.48	81%	0.19	76%	0.17	81%	0.03	75%
3.00	0.55	83%	0.25	89%	0.40	85%	0.14	78%	0.66	86%	0.30	83%	0.50	85%	0.20	80%	0.18	86%	0.03	75%
3.50	0.57	86%	0.25	89%	0.41	87%	0.15	83%	0.67	87%	0.31	86%	0.51	86%	0.21	84%	0.18	86%	0.03	75%
4.00	0.58	88%	0.26	93%	0.41	87%	0.16	89%	0.69	90%	0.32	89%	0.52	88%	0.21	84%	0.19	90%	0.03	75%
Ultimate Settlement due to Primary Consolidation	0.66	100%	0.28	100%	0.47	100%	0.18	100%	0.77	100%	0.36	100%	0.59	100%	0.25	100%	0.21	100%	0.04	100%

NBL - NORTH APPROACH-1 (10+620 to 10+675)



NBL

TABLE F6

HIGHWAY 69 - FOUR LANING - CNR CROSSING NORTH OF ESTAIRE  
SUMMARY OF SETTLEMENT DUE TO PRIMARY CONSOLIDATION  
NORTHBOUND LANE BRIDGE (NBL) - NORTH APPROACH-2 (10+675 to Dill TWP 10+010)

Case A: 3.8m Rockfill Embankment + 3m Surcharge  
Case B: 3.8m Rockfill Embankment  
Case C: 3.8m Sandfill Embankment + 3m Surcharge  
Case D: 3.8m Sandfill Embankment  
Case E: 3.8m REP Embankment and Sandfill Cover

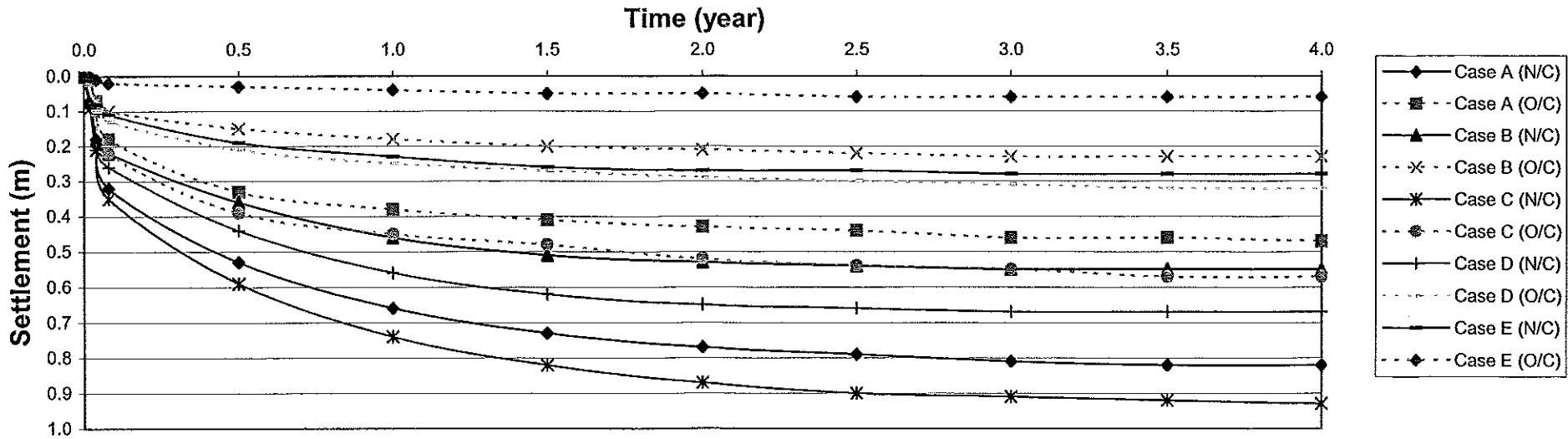
Note

1. N/C: Normal Consolidated

2. O/C: Over Consolidated

Time from Beginning of Construction (yr)	NBL - NORTH APPROACH-2 (10+675 to Dill TWP 10+010)																			
	Settlement (m) and % of Consolidation at centerline of NBL																			
	Case A				Case B				Case C				Case D				Case E			
	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%
0.00	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%
0.02	0.07	9%	0.01	2%	0.07	13%	0.01	4%	0.09	9%	0.01	2%	0.09	13%	0.01	3%	0.01	4%	0.00	0%
0.04	0.18	22%	0.07	15%	0.18	33%	0.07	27%	0.21	21%	0.10	16%	0.21	31%	0.10	28%	0.08	29%	0.01	14%
0.08	0.32	39%	0.18	38%	0.22	40%	0.10	38%	0.35	35%	0.22	34%	0.26	39%	0.13	36%	0.11	39%	0.02	29%
0.50	0.53	65%	0.33	70%	0.36	65%	0.15	58%	0.59	58%	0.39	61%	0.44	66%	0.21	58%	0.19	68%	0.03	43%
1.00	0.66	80%	0.38	81%	0.46	84%	0.18	69%	0.74	73%	0.45	70%	0.56	84%	0.25	69%	0.23	82%	0.04	57%
1.50	0.73	89%	0.41	87%	0.51	93%	0.20	77%	0.82	81%	0.48	75%	0.62	93%	0.27	75%	0.26	93%	0.05	71%
2.00	0.77	94%	0.43	91%	0.53	96%	0.21	81%	0.87	86%	0.52	81%	0.65	97%	0.29	81%	0.27	96%	0.05	71%
2.50	0.79	96%	0.44	94%	0.54	98%	0.22	85%	0.90	89%	0.54	84%	0.66	99%	0.30	83%	0.27	96%	0.06	86%
3.00	0.81	99%	0.46	98%	0.55	100%	0.23	88%	0.91	90%	0.55	86%	0.67	100%	0.31	86%	0.28	100%	0.06	86%
3.50	0.82	100%	0.46	98%	0.55	100%	0.23	88%	0.92	91%	0.57	89%	0.67	100%	0.32	89%	0.28	100%	0.06	86%
4.00	0.82	100%	0.47	100%	0.55	100%	0.23	88%	0.93	92%	0.57	89%	0.67	100%	0.32	89%	0.28	100%	0.06	86%
Ultimate Settlement due to Primary Consolidation	0.82	100%	0.47	100%	0.55	100%	0.26	100%	1.01	100%	0.64	100%	0.67	100%	0.36	100%	0.28	100%	0.07	100%

NBL - NORTH APPROACH-2 (10+675 to Dill TWP 10+010)



NBL

TABLE F7

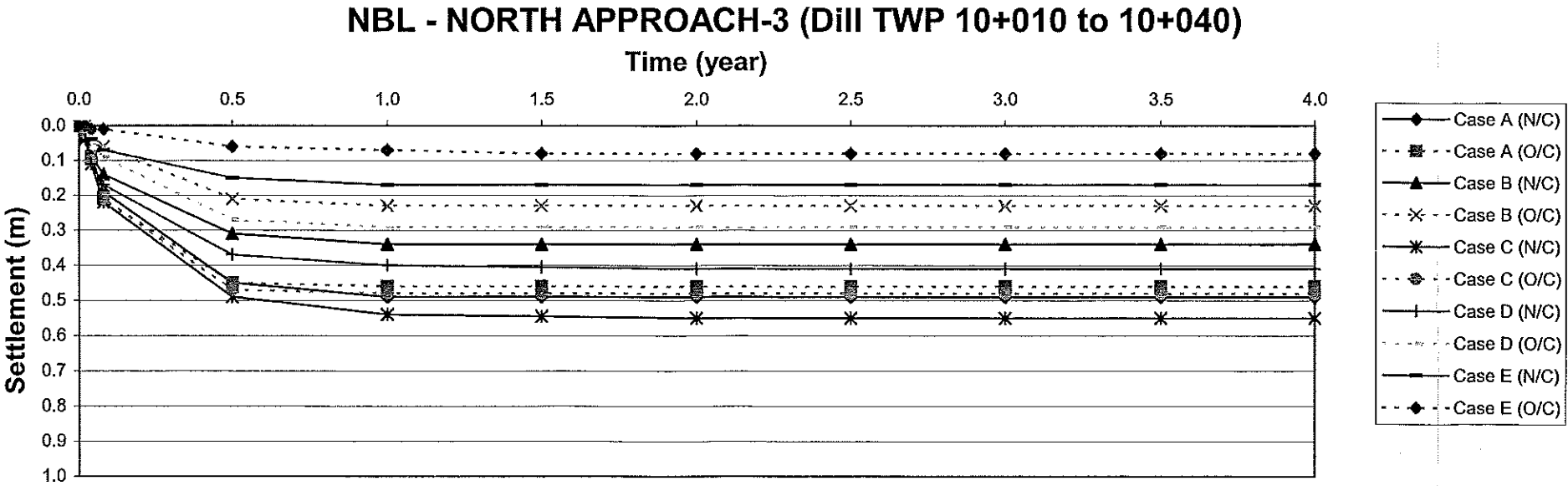
**HIGHWAY 69 - FOUR LANING - CNR CROSSING NORTH OF ESTAIRE**  
**SUMMARY OF SETTLEMENT DUE TO PRIMARY CONSOLIDATION**  
**NORTHBOUND LANE BRIDGE (NBL) - NORTH APPROACH-3 (DIII TWP 10+010 to 10+040)**

**Case A:** 4.2m Rockfill Embankment + 3m Surcharge  
**Case B:** 4.2m Rockfill Embankment  
**Case C:** 4.2m Sandfill Embankment + 3m Surcharge  
**Case D:** 4.2m Sandfill Embankment  
**Case E:** 4.2m REP Embankment and Sandfill Cover

Note

1. N/C: Normal Consolidated
2. O/C: Over Consolidated

Time from Beginning of Construction (yr)	NBL - NORTH APPROACH-3 (DIII TWP 10+010 to 10+040)																			
	Settlement (m) and % of Consolidation at centerline of NBL																			
	Case A				Case B				Case C				Case D				Case E			
	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%	N/C	%	O/C	%
0.00	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%	0.00	0%
0.02	0.03	6%	0.02	4%	0.03	9%	0.00	0%	0.04	7%	0.02	4%	0.04	10%	0.01	3%	0.00	0%	0.00	0%
0.04	0.09	18%	0.09	20%	0.09	26%	0.03	13%	0.11	20%	0.10	21%	0.11	27%	0.05	17%	0.04	24%	0.01	13%
0.08	0.19	39%	0.21	46%	0.14	41%	0.06	26%	0.22	40%	0.21	44%	0.17	41%	0.09	31%	0.07	41%	0.01	13%
0.50	0.45	92%	0.45	98%	0.31	91%	0.21	91%	0.49	89%	0.47	98%	0.37	90%	0.27	93%	0.15	88%	0.06	75%
1.00	0.49	100%	0.46	100%	0.34	100%	0.23	100%	0.54	98%	0.48	100%	0.40	98%	0.29	100%	0.17	100%	0.07	88%
1.50	0.49	100%	0.46	100%	0.34	100%	0.23	100%	0.55	99%	0.48	100%	0.41	99%	0.29	100%	0.17	100%	0.08	100%
2.00	0.49	100%	0.46	100%	0.34	100%	0.23	100%	0.55	100%	0.48	100%	0.41	100%	0.29	100%	0.17	100%	0.08	100%
2.50	0.49	100%	0.46	100%	0.34	100%	0.23	100%	0.55	100%	0.48	100%	0.41	100%	0.29	100%	0.17	100%	0.08	100%
3.00	0.49	100%	0.46	100%	0.34	100%	0.23	100%	0.55	100%	0.48	100%	0.41	100%	0.29	100%	0.17	100%	0.08	100%
3.50	0.49	100%	0.46	100%	0.34	100%	0.23	100%	0.55	100%	0.48	100%	0.41	100%	0.29	100%	0.17	100%	0.08	100%
4.00	0.49	100%	0.46	100%	0.34	100%	0.23	100%	0.55	100%	0.48	100%	0.41	100%	0.29	100%	0.17	100%	0.08	100%
Ultimate Settlement due to Primary Consolidation	0.49	100%	0.46	100%	0.34	100%	0.23	100%	0.55	100%	0.48	100%	0.41	100%	0.29	100%	0.17	100%	0.08	100%



## Appendix G

### Wick Drain Analysis Results

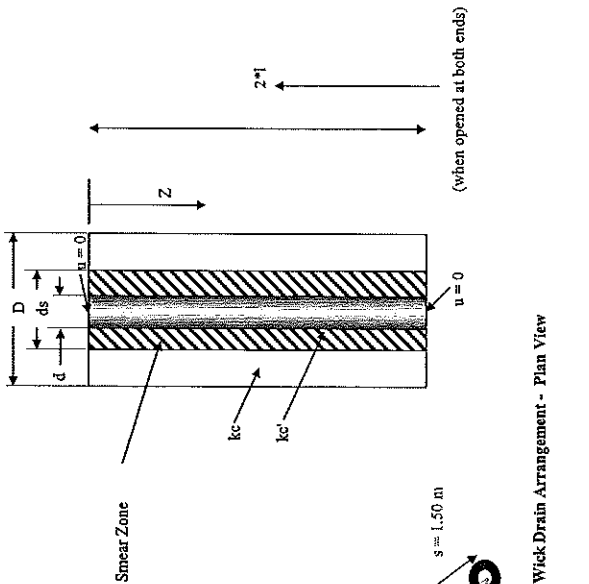
**NEW HANSBO METHOD (combined with Lambe & Whitman's book) recommendations**  
**"Consolidation of Clay by Band-Shaped Prefabricated Drains"**  
**Ground Engineering, Vol.12 No.5, 1979**  
**Formulation according to Equation 1 - Including well resistance and smearing**

Job Number: 15-64-15  
 Title: Hvy69 - Estaire - 4-Laning  
 Case: SBL - North Abutment  
 Sub-case: Case C

**INPUT PARAMETERS**

D	1.575	m	diameter of dewatered soil cylinder (Triangular Spacing equal to, $s =$	1.50	m)
d	0.07	m	equivalent diameter of band-shaped drain: $2(0+4)/\pi$ ; $n =$	22.5	
Cu	6.43E-07	m <sup>2</sup> /s	consider reducing $c_v$ to account for smear; $Cu/Cv$ is often 2 to 5		
Cv	6.43E-07	m <sup>2</sup> /s	determined by the oedometer test		
$\lambda$	6.43E-07	m <sup>2</sup> /s	$=k_v/(q_w \cdot m)$ ; or $\lambda = Cv$ obtained from the oedometer test (Hansbo 1979)		
$d_s$	0.21	m	diameter of the smear zone (typically equal to 1.5 to 3 times $d$ ); $s=d_s/d =$	3	
$k_v$	1.00E-10	m/s	undisturbed soil permeability		
$k'_c$	3.33E-11	m/s	soil permeability within the smear zone; $k_c/k'_c =$	3.00	
$q_w$	1.00E-05	m <sup>3</sup> /s	drain discharge capacity; $k_c/q_w =$	1.00E-05	; well resistance cannot be ignored if $k_c/q_w > 3.33E-04$
$l$	17.00	m	length of the drain when open at one end only		
Wick drainage (one end; 1; two ends; 2):	2		half length of the drain when open at both ends		
Layer	Cl-CH				
Surcharge (kPa)	190.80	kPa			
Drainage Path (m)	17.00	m			
Settlement due to Primary Consolidation	1370	mm			
$\alpha$	23	(D/d; should always be >12)			
$\alpha$	0.2850248	f(D/d); regression from Figure 3 of the paper)			

Time Increment for table below = 0.11 month  
 Resultant Maximum Time = 6.71 months



Wick Drain Arrangement - Plan View

% Consolidation	Time required (months)	
	Uv and Uh	Uh only
16	0.22	0.22
25	0.22	0.33
90	1.87	1.98
98	3.19	3.30



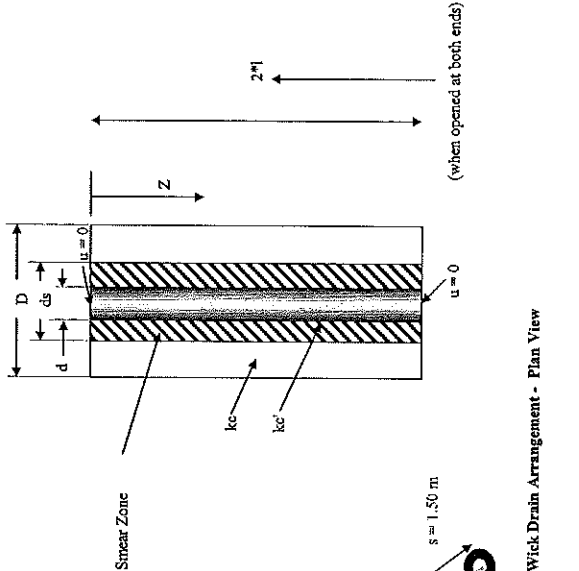
NEW HANSBO METHOD (combined with Lambe & Whitman's book recommendations  
"Consolidation of Clay by Band-Shaped Prefabricated Drains"  
Ground Engineering, Vol.12 No.5, 1979  
Formulation according to Equation 1 - Including well resistance and smearing

Job Number: 15-64-15  
Title: Hwy69 - Estaire - 4-Laning  
Case: NBL - North Abutment (10+580 to 10+620)  
Sub-case: Case C

INPUT PARAMETERS

D	1.575	m	diameter of dewatered soil cylinder (Triangular Spacing equal to, $s =$	1.50	m)
d	0.07	m	equivalent diameter of band-shaped drain: $2(b+t)/\pi$ ; $n =$	22.5	
C <sub>tr</sub>	6.43E-07	m <sup>2</sup> /s	consider reducing $c_v$ to account for smear; $Cb/Cv$ is often 2 to 5		
C <sub>v</sub>	6.43E-07	m <sup>2</sup> /s	determined by the oedometer test		
$\lambda$	6.43E-07	m <sup>2</sup> /s	$=k_v/(v_w \cdot n_b)$ , or $\lambda = Cv$ obtained from the oedometer test (Hansbo 1979)		
d <sub>1</sub>	0.21	m	diameter of the smear zone (typically equal to 1.5 to 3 times d); $s=ds/d =$	3	
k <sub>c</sub>	1.00E-10	m/s	undisturbed soil permeability		
k <sub>c'</sub>	3.33E-11	m/s	soil permeability within the smear zone; $k_c/k_{c'} =$	3.00	
q <sub>w</sub>	1.20E-05	m <sup>3</sup> /s	drain discharge capacity; $k_c/q_w =$	8.33E-06	well resistance cannot be ignored if $k_c/q_w > 3.33E-04$
l	14.50	m	length of the drain when open at one end only		
Wick drainage (one end:1; two ends:2):	2		half length of the drain when open at both ends		
Layer	CL-CH				
Surcharge (kPa)	204.60	kPa			
Drainage Path (m)	14.50	m			
Sediment due to Primary Consolidation	1050	mm			
n	23	(D/d); should always be >12)			
$\alpha$	0.2850248	f(D/d); regression from Figure 3 of the paper			

Time Increment for table below = 0.15 month  
Resultant Maximum Time = 9.15 months



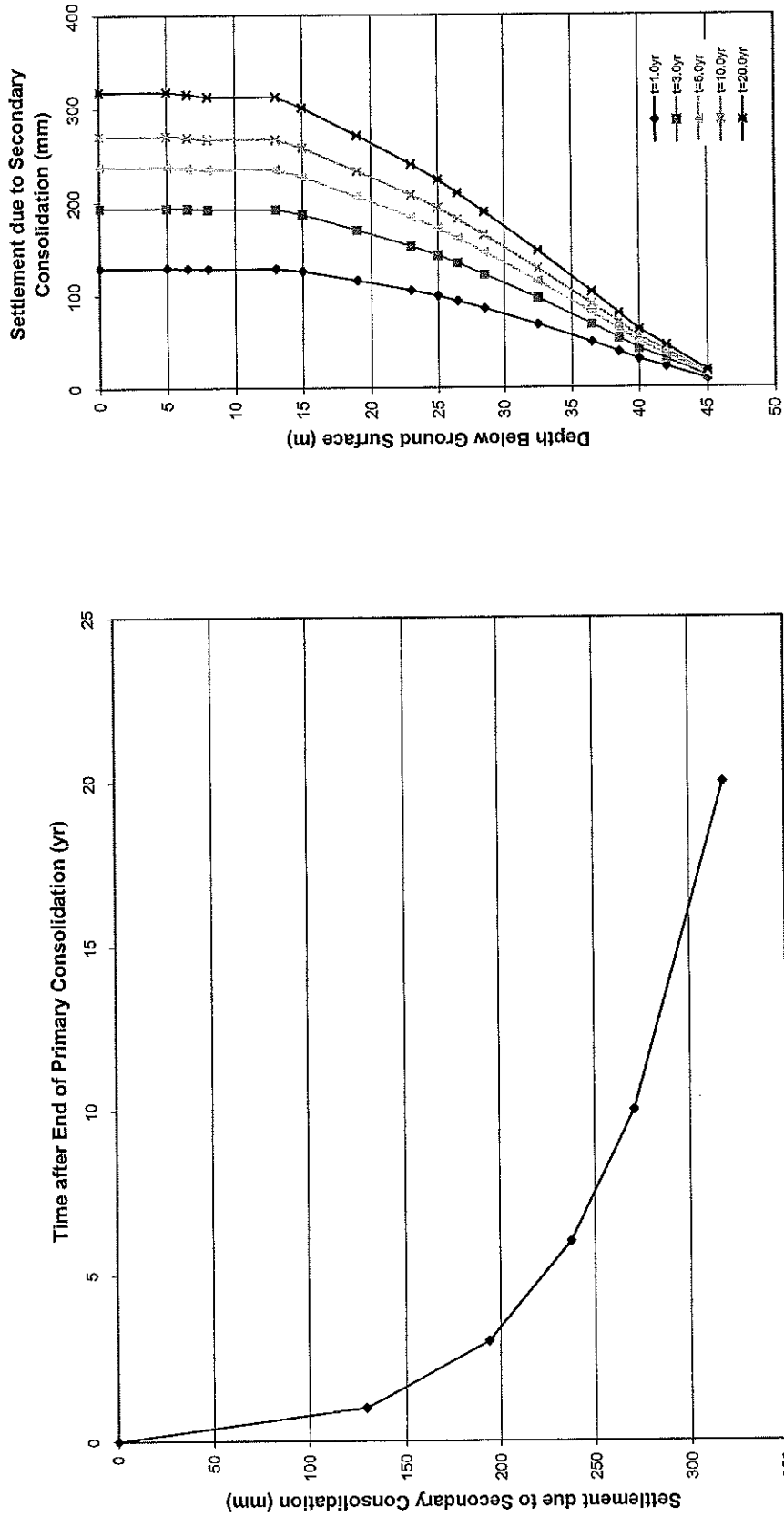
% Consolidation	Time required (months)	
	U <sub>v</sub> and U <sub>h</sub>	U <sub>h</sub> only
16	0.15	0.15
25	0.30	0.30
90	1.95	1.95
98	3.15	3.30

**Appendix H**  
**Secondary Consolidation Analysis Results**

**Secondary Consolidation Analysis**  
**SBL Approach Embankment**

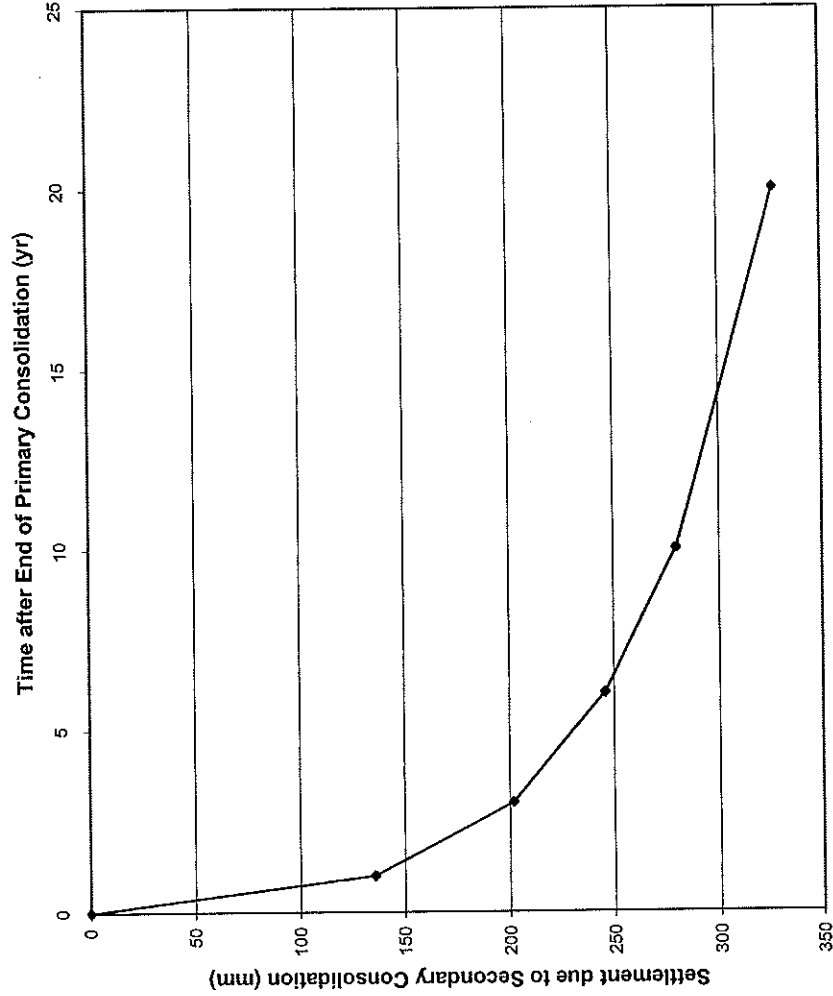
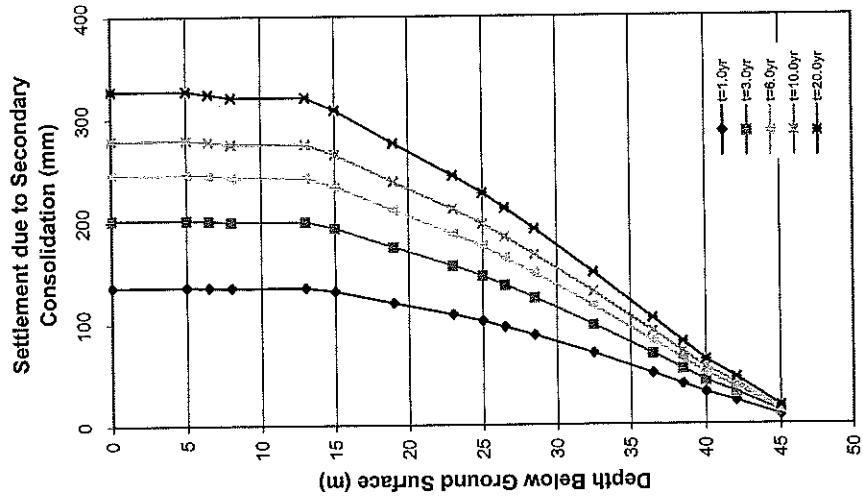
HIGHWAY 69 - FOUR LANING - CNR CROSSING NORTH OF ESTAIRE  
SETTLEMENT DUE TO SECONDARY CONSOLIDATION  
SOUTHBOUND LANE BRIDGE (SBL) - North Abutment (10+550 to 10+600)

SBL North Abutment - [Rockfill + Surcharge] to Rockfill - Case A to Case B - O/C

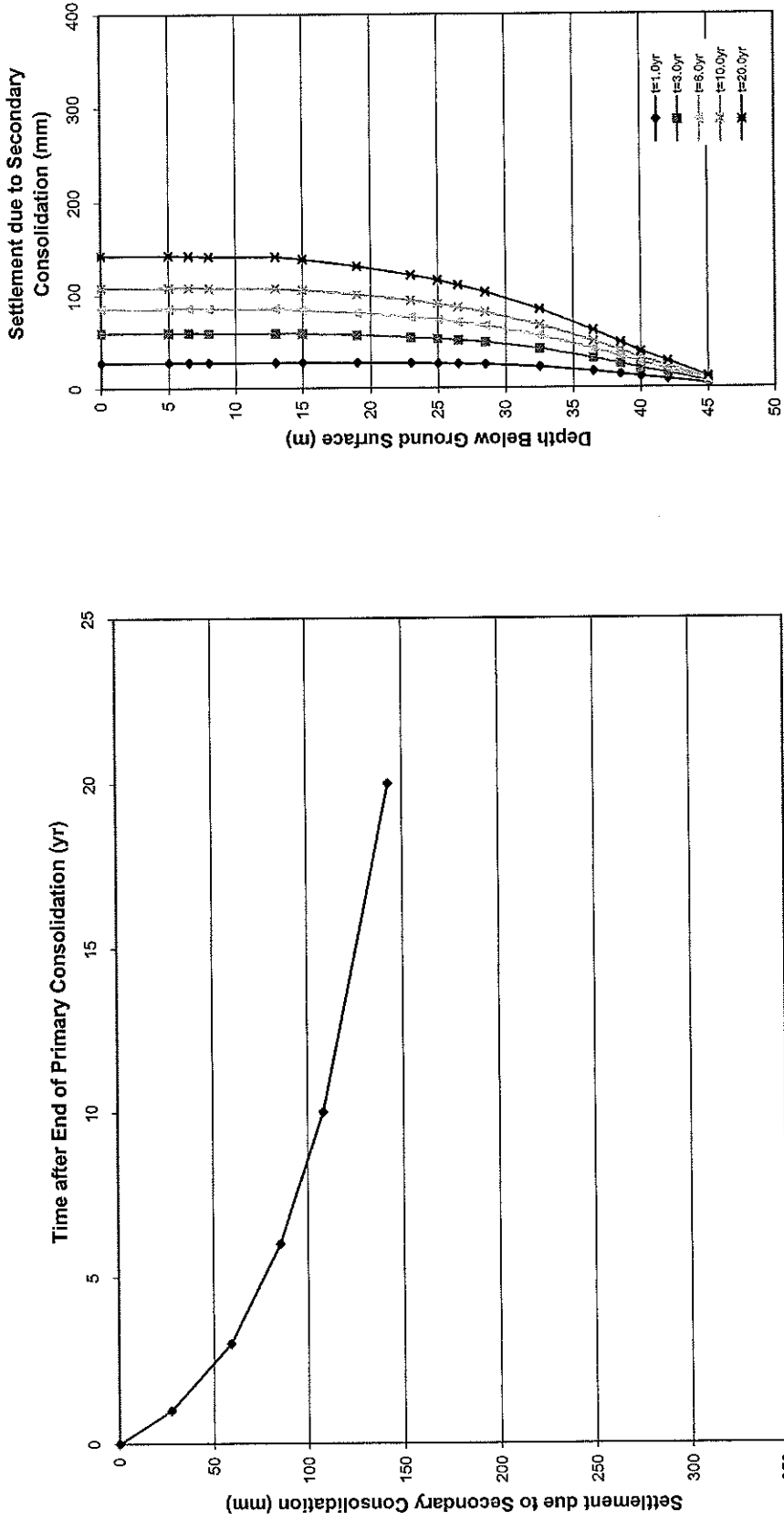


HIGHWAY 69 - FOUR LANE - CNR CROSSING NORTH OF ESTAIRE  
SETTLEMENT DUE TO SECONDARY CONSOLIDATION  
SOUTHBOUND LANE BRIDGE (SBL) - North Abutment (10+550 to 10+600)

SBL North Abutment - [Sandfill + Surcharge] to Sandfill - Case C to Case D - O/C

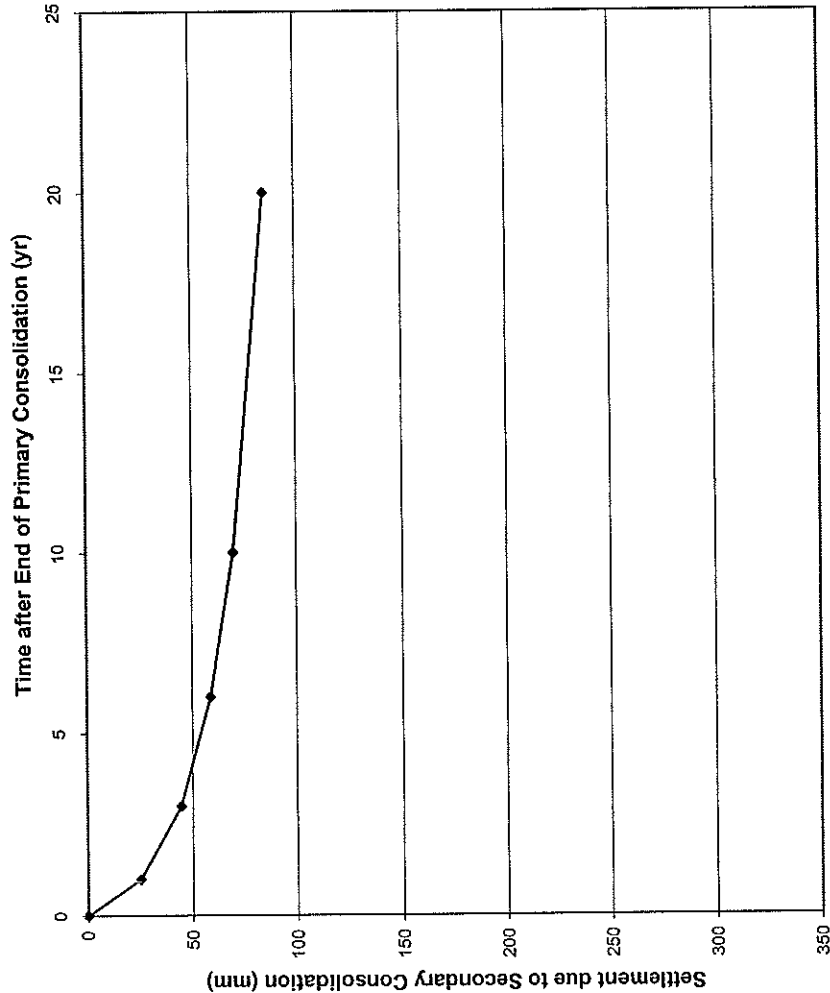
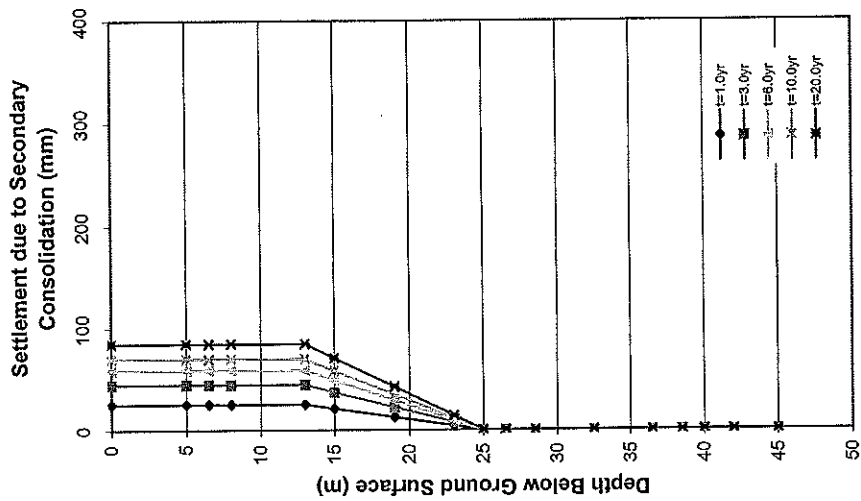


HIGHWAY 69 - FOUR LANE - CNR CROSSING NORTH OF ESTAIRE  
SETTLEMENT DUE TO SECONDARY CONSOLIDATION  
SOUTHBOUND LANE BRIDGE (SBL) - North Abutment (10+550 to 10+600)  
  
SBL North Abutment - Sandfill to Geofoam - Case D to E - O/C



HIGHWAY 69 - FOUR LANE - CNR CROSSING NORTH OF ESTAIRE  
SETTLEMENT DUE TO SECONDARY CONSOLIDATION  
SOUTHBOUND LANE BRIDGE (SBL) - North Abutment (10+550 to 10+600)

SBL North Abutment - Geofoam - Case E to E- O/C



Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: SBL North Abutment - [Rockfill + Surcharge] to Rockfill - Case A to Case B - N/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs' ( $\sigma_v'/\sigma_v$ )-1	Cas	Time after end of Primary Consolidation (years)						
		From	To		Vertical Eff. Stress (kPa)		Settlement due to Secondary Consolidation (mm)	1			3	6	10	20			
					$\sigma_v$ with surcharge	$\sigma_v'$ w/o surcharge											
1	Clayey Silt	5	6.5	1.5	249.72	202.25	0.23470952	0.0026	0.2	0.7	1.2	1.7	2.5				
2	Clayey Silt	6.5	8	1.5	255.85	212.96	0.20139932	0.0026	0.2	0.9	1.5	2.0	2.9				
3	SP&ML (NP)	8	13	5	280.46	235.96	0.18859129	0	0.0	0.0	0.0	0.0	0.0				
4	CL-Cl	13	15	2	292.09	259.29	0.12649929	0.0044	2.8	5.5	7.5	9.1	11.4				
5	CL-Cl	15	19	4	301.07	277.85	0.09357027	0.0044	10.1	16.8	21.3	24.9	30.0				
6	CL-Cl	19	23	4	327.93	303.82	0.0793562	0.0044	10.7	17.4	22.1	25.7	30.8				
7	CL-Cl	23	25	2	346.02	323.98	0.06802889	0.0044	6.2	9.7	12.1	13.9	16.5				
8	Cl-CH	25	26.5	1.5	356.12	335.29	0.06212532	0.005	5.6	8.7	10.8	12.4	14.6				
9	Cl-CH	26.5	28.5	2	365.63	345.82	0.05728414	0.005	8.0	12.2	15.0	17.1	20.1				
10	Cl-CH	28.5	32.5	4	382.65	364.36	0.05019761	0.005	17.6	26.1	31.8	36.1	42.1				
11	Cl-CH	32.5	36.5	4	406.26	389.81	0.04220005	0.005	19.5	28.3	34.1	38.5	44.5				
12	Cl-CH	36.5	38.5	2	424.54	409.37	0.03705694	0.005	10.5	15.0	17.9	20.1	23.1				
13	Cl-CH	38.5	40	1.5	435.52	420.98	0.03453846	0.005	8.2	11.5	13.7	15.4	17.7				
14	CL-ML	40	42	2	447.91	434.21	0.03155155	0.0036	8.2	11.5	13.6	15.2	17.4				
15	CL-ML	42	45	3	466.41	454.91	0.02527973	0.0036	13.6	18.6	21.8	24.2	27.5				
16	CL-ML	45	47	2	489.29	476.14	0.02761793	0.0036	8.7	12.0	14.1	15.7	17.9				
									130.2	195.0	238.5	271.9	318.9				



Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: SBL North Abutment - [Rockfill + Surcharge] to Rockfill - Case A to Case B - O/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs <sup>1</sup> ( $\sigma_{vs}/\sigma_{vf}$ )-1	Case	Time after end of Primary Consolidation (Years)					
		From	To		With surcharge	σ <sub>vs</sub>	σ <sub>vs</sub>	σ <sub>vf</sub>			1	3	6	10	20	
1	Clayey Silt	5	6.5	1.5	248.35	201.48	212.47	0.23282855	0.0026	0.0026	0.2	0.7	1.2	1.7	2.5	
2	Clayey Silt	6.5	8	1.5	255.19	212.47	0.20106368	0.0026	0.0026	0.0026	0.2	0.9	1.5	2.0	2.9	
3	SP&ML (NP)	8	13	5	278.58	235.25	0.18418704	0	0.0044	0	0.0	0.0	0.0	0.0	0.0	
4	CL-Cl	13	15	2	291.09	258.66	0.12537694	0.0044	0.0044	0.0044	2.9	5.6	7.6	9.2	11.5	
5	CL-Cl	15	19	4	300.76	277.56	0.08358553	0.0044	0.0044	0.0044	10.1	16.8	21.3	24.9	30.0	
6	CL-Cl	19	23	4	327.72	303.63	0.07933999	0.0044	0.0044	0.0044	10.7	17.5	22.1	25.7	30.8	
7	CL-Cl	23	25	2	346.01	323.86	0.06893975	0.0044	0.0044	0.0044	8.1	9.7	12.0	13.9	16.5	
8	CL-CH	25	26.5	1.5	356.18	335.2	0.0625895	0.005	0.005	0.005	5.6	8.7	10.8	12.3	14.6	
9	CL-CH	26.5	28.5	2	365.75	345.77	0.05778408	0.005	0.005	0.005	8.0	12.1	14.9	17.0	20.0	
10	CL-CH	28.5	32.5	4	382.85	364.35	0.05077535	0.005	0.005	0.005	17.4	26.0	31.7	36.0	42.0	
11	CL-CH	32.5	36.5	4	406.56	389.86	0.04283589	0.005	0.005	0.005	19.4	28.2	33.9	38.3	44.3	
12	CL-CH	36.5	38.5	2	424.9	409.46	0.0377082	0.005	0.005	0.005	10.4	14.9	17.8	20.0	23.0	
13	CL-CH	38.5	40	1.5	435.9	421.08	0.03519521	0.005	0.005	0.005	8.1	11.5	13.7	15.3	17.6	
14	CL-ML	40	42	2	448.27	434.33	0.03209541	0.0036	0.0036	0.0036	8.1	11.4	13.5	15.1	17.3	
15	CL-ML	42	45	3	466.96	455.05	0.02617295	0.0036	0.0036	0.0036	13.4	18.4	21.6	24.0	27.3	
16	CL-ML	45	47	2	489.19	476.03	0.02764532	0.0036	0.0036	0.0036	8.7	12.0	14.1	15.7	17.9	
											129.4	194.2	237.6	271.0	318.0	

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: SBL North Abutment - [Sandfill + Surcharge] to Sandfill - Case C to Case D - N/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs ( $\sigma_v/\sigma_v'$ )-1	Cos	Time after end of Primary Consolidation (years)					
		From	To		$\sigma_v'$ with surcharge	Vertical Eff. Stress (kPa) $\sigma_v'$	$\sigma_v'$ w/o surcharge	$\sigma_v'$			Settlement due to Secondary Consolidation (mm)					
1	Clayey Silt	5	6.5	1.5	273.77	226.1	226.1	0.21083591	0.0026	0.0026	0.2	0.8	1.4	1.9	2.5	2.7
2	Clayey Silt	6.5	8	1.5	279.76	236.91	236.91	0.18087037	0.0026	0.0026	0.5	1.2	1.9	2.5	3.4	3.4
3	SP&ML (NP)	8	13	5	306.44	260.19	260.19	0.17775472	0	0	0.0	0.0	0.0	0.0	0.0	0.0
4	CL-Cl	13	15	2	316.4	283.33	283.33	0.11671902	0.0044	0.0044	3.3	6.1	8.1	9.8	12.2	12.2
5	CL-Cl	15	19	4	323.17	301.3	301.3	0.07258546	0.0044	0.0044	11.6	18.6	23.3	26.9	32.1	32.1
6	CL-Cl	19	23	4	350.23	326.29	326.29	0.07337031	0.0044	0.0044	11.5	18.4	23.2	26.8	31.9	31.9
7	CL-Cl	23	25	2	367.57	345.58	345.58	0.06363215	0.0044	0.0044	6.5	10.1	12.5	14.4	17.0	17.0
8	CL-CH	25	26.5	1.5	377.12	356.34	356.34	0.05831509	0.005	0.005	5.9	9.1	11.1	12.7	15.0	15.0
9	CL-CH	26.5	28.5	2	386.06	366.31	366.31	0.0474888	0.005	0.005	8.4	12.6	15.4	17.6	20.5	20.5
10	CL-CH	28.5	32.5	4	402.11	383.88	383.88	0.0474888	0.005	0.005	18.2	26.9	32.6	36.9	42.9	42.9
11	CL-CH	32.5	36.5	4	424.47	408.09	408.09	0.0401382	0.005	0.005	20.1	29.0	34.8	39.1	45.2	45.2
12	CL-CH	36.5	38.5	2	441.84	426.77	426.77	0.03531176	0.005	0.005	10.8	15.3	18.2	20.4	23.4	23.4
13	CL-CH	38.5	40	1.5	452.32	437.88	437.88	0.03297707	0.005	0.005	8.4	11.8	14.0	15.6	17.9	17.9
14	CL-ML	40	42	2	464.11	450.63	450.63	0.02991368	0.0036	0.0036	8.4	11.7	13.8	15.4	17.6	17.6
15	CL-ML	42	45	3	481.35	470.68	470.68	0.02266933	0.0036	0.0036	14.2	19.2	22.4	24.8	28.1	28.1
16	CL-ML	45	47	2	504.53	491.38	491.38	0.02676137	0.0036	0.0036	8.8	12.1	14.3	15.9	18.1	18.1
											136.8	202.9	247.0	280.7	328.0	328.0

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: SBL North Abutment - [Sandfill + Surcharge] to Sandfill - Case C to Case D - O/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs (σ <sub>vs</sub> /σ <sub>vf</sub> )-1	C <sub>as</sub>	Time after end of Primary Consolidation (years)					
		From	To		Vertical Eff. Stress (kPa)		σ <sub>vf</sub>				1	3	6	10	20	
					with surcharge	w/o surcharge	σ <sub>vs</sub>	σ <sub>vf</sub>								
Settlement due to Secondary Consolidation (mm)																
1	Clayey Silt	5	6.5	1.5	272.1	224.93	0.20970969	0.0026	0.2	0.8	1.4	1.9	2.8			
2	Clayey Silt	6.5	8	1.5	278.84	236.07	0.18117508	0.0026	0.5	1.2	1.9	2.5	3.4			
3	SP&ML (NP)	8	13	5	304.11	259.08	0.17380732	0	0.0	0.0	0.0	0.0	0.0			
4	CL-Cl	13	15	2	315.08	282.33	0.11599901	0.0044	3.3	6.1	8.2	9.8	12.2			
5	CL-Cl	15	19	4	322.57	300.71	0.07269462	0.0044	11.6	18.6	23.3	26.9	32.1			
6	CL-Cl	19	23	4	349.73	325.83	0.07335113	0.0044	11.5	18.5	23.2	26.8	31.9			
7	CL-Cl	23	25	2	367.3	345.21	0.06399004	0.0044	6.5	10.1	12.5	14.3	16.9			
8	Cl-CH	25	26.5	1.5	376.93	356.02	0.05873266	0.005	5.9	9.0	11.1	12.7	14.9			
9	Cl-CH	26.5	28.5	2	385.94	366.04	0.05436564	0.005	8.3	12.5	15.4	17.5	20.5			
10	Cl-CH	28.5	32.5	4	402.09	383.67	0.04801001	0.005	18.1	26.7	32.4	36.7	42.8			
11	Cl-CH	32.5	36.5	4	424.56	407.96	0.04069026	0.005	20.0	28.8	34.6	38.9	45.0			
12	Cl-CH	36.5	38.5	2	442	426.69	0.03588085	0.005	10.7	15.2	18.1	20.3	23.3			
13	Cl-CH	38.5	40	1.5	452.51	437.83	0.033529	0.005	8.3	11.7	13.9	15.5	17.8			
14	CL-ML	40	42	2	464.28	450.61	0.03033665	0.0036	8.4	11.6	13.8	15.3	17.5			
15	CL-ML	42	45	3	481.79	470.7	0.02356065	0.0036	14.0	19.0	22.2	24.6	27.9			
16	CL-ML	45	47	2	504.26	491.1	0.02679699	0.0036	8.8	12.1	14.3	15.9	18.1			
									136.0	202.0	246.1	279.8	327.1			

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: SBL North Abutment - Sandfill to Geofoam - Case D to E - N/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=6m)		Rs (σ <sub>v</sub> /σ <sub>vh</sub> )	Cos	Time after end of Primary Consolidation (years)						
		From	To		Vertical Eff. Stress (kPa)	σ <sub>v</sub>			Settlement due to Secondary Consolidation (mm)						
					with surcharge	σ <sub>v</sub> w/o surcharge			1	3	5	10	20		
1	Clayey Silt	5	6.5	1.5	226.1	130.82	0.728329	0.0026	0.0	0.0	0.2	0.3	0.6		
2	Clayey Silt	6.5	8	1.5	236.91	143.58	0.65002089	0.0026	0.0	0.1	0.2	0.4	0.7		
3	SP&ML (NP)	8	13	5	260.19	171.1	0.52068866	0	0.0	0.0	0.0	0.0	0.0		
4	CL-CL	13	15	2	283.33	200.05	0.41829593	0.0044	0.0	0.6	1.2	1.9	3.0		
5	CL-CL	15	19	4	301.3	223.78	0.34841165	0.0044	0.2	1.7	3.3	4.9	7.4		
6	CL-CL	19	23	4	326.29	255.72	0.2759659	0.0044	0.5	2.5	4.5	6.4	9.5		
7	CL-CL	23	25	2	345.58	279.87	0.23478758	0.0044	0.4	1.6	2.8	3.9	5.6		
8	CL-CH	25	26.5	1.5	356.34	293.29	0.21497494	0.005	0.4	1.5	2.6	3.6	5.2		
9	CL-CH	26.5	28.5	2	366.31	305.79	0.1979136	0.005	0.7	2.3	3.9	5.3	7.5		
10	CL-CH	28.5	32.5	4	383.88	327.34	0.17272561	0.005	2.8	7.2	10.8	13.9	18.6		
11	CL-CH	32.5	36.5	4	408.09	356.3	0.14535504	0.005	4.8	10.2	14.3	17.7	22.8		
12	CL-CH	36.5	38.5	2	426.77	378.18	0.12848379	0.005	3.1	6.1	8.4	10.2	12.8		
13	CL-CH	38.5	40	1.5	437.88	391	0.1198977	0.005	2.7	5.0	6.8	8.1	10.2		
14	CL-ML	40	42	2	450.63	405.35	0.11170593	0.0036	2.9	5.2	7.0	8.3	10.3		
15	CL-ML	42	45	3	470.68	427.53	0.10082859	0.0036	5.0	8.7	11.4	13.5	16.5		
16	CL-ML	45	47	2	491.38	449.82	0.09239251	0.0036	3.7	6.3	8.1	9.6	11.6		
									27.3	59.2	85.5	107.9	142.4		

**Job Number:** 15-64-15  
**Job Description:** Hwy69 - Four-Laning - Estaire  
**Case:** SBL North Abutment - Sandfill to Geofoam - Case D to E - O/C

**Duration of Primary Consolidation (tp) (years):** 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=6m)		Rs ( $\sigma_v/\sigma_v'-1$ )	Cos	Time after end of Primary Consolidation (Years)				
		From	To		Vertical Eff. Stress (kPa)				Settlement due to Secondary Consolidation (mm)				
					$\sigma_v$ with surcharge	$\sigma_v'$ w/o surcharge			1	3	6	10	20
1	Clayey Silt	5	6.5	1.5	224.93	130.66	0.72149089	0.0026	0.0	0.0	0.2	0.3	0.6
2	Clayey Silt	6.5	8	1.5	236.07	143.47	0.6454311	0.0026	0.0	0.1	0.2	0.4	0.7
3	SP&ML (NP)	8	13	5	259.08	170.96	0.51544221	0	0.0	0.0	0.0	0.0	0.0
4	CL-Cl	13	15	2	282.33	199.92	0.41221489	0.0044	0.0	0.6	1.3	1.9	3.0
5	CL-Cl	15	19	4	300.71	223.71	0.34419561	0.0044	0.2	1.7	3.3	4.9	7.5
6	CL-Cl	19	23	4	325.83	255.66	0.27446609	0.0044	0.5	2.5	4.5	6.4	9.6
7	CL-Cl	23	25	2	345.21	279.82	0.23368594	0.0044	0.4	1.6	2.8	3.9	5.6
8	Cl-CH	25	26.5	1.5	356.02	293.26	0.21400805	0.005	0.4	1.5	2.6	3.6	5.2
9	Cl-CH	26.5	28.5	2	366.04	305.75	0.19718724	0.005	0.7	2.4	3.9	5.3	7.6
10	Cl-CH	28.5	32.5	4	383.67	327.31	0.1721915	0.005	2.9	7.2	10.8	13.9	18.7
11	Cl-CH	32.5	36.5	4	407.96	356.28	0.14505445	0.005	4.8	10.2	14.4	17.8	22.9
12	Cl-CH	36.5	38.5	2	426.69	378.17	0.12830209	0.005	3.2	6.2	8.4	10.2	12.8
13	Cl-CH	38.5	40	1.5	437.83	390.99	0.11979846	0.005	2.7	5.0	6.8	8.1	10.2
14	CL-ML	40	42	2	450.61	405.35	0.11165659	0.0036	2.9	5.3	7.0	8.3	10.3
15	CL-ML	42	45	3	470.7	427.53	0.10097537	0.0036	5.0	8.7	11.4	13.5	16.5
16	CL-ML	45	47	2	491.1	449.78	0.09186714	0.0036	3.7	6.4	8.2	9.6	11.6
									27.4	59.4	85.8	108.2	142.8

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: SBL North Abutment - Geofoam - Case E to E - N/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs ( $\sigma'vs\sigma'f-1$ )	C <sub>as</sub>	Time after end of Primary Consolidation (years)					
		From	To		$\sigma'vs$ With surcharge	Vertical Eff. Stress (kPa)	$\sigma'vs$ w/o surcharge	$\sigma'vs$			1	3	6	10	20	
1	Clayey Silt	5	6.5	1.5	130.82	130.82	130.82	0	0.0026	0	1.9	3.3	4.3	5.2	6.3	
2	Clayey Silt	6.5	8	1.5	143.58	143.58	143.58	0	0.0026	0	1.9	3.3	4.3	5.2	6.3	
3	SP&ML (NP)	8	13	5	171.1	171.1	171.1	0	0	0	0.0	0.0	0.0	0.0	0.0	
4	CL-Cl	13	15	2	200.05	200.05	200.05	0	0.0044	0	4.2	7.4	9.8	11.6	14.2	
5	CL-Cl	15	19	4	223.78	223.78	223.78	0	0.0044	0	8.4	14.9	19.6	23.3	28.4	
6	CL-Cl	19	23	4	255.72	255.72	255.72	0	0.0044	0	8.4	14.9	19.6	23.3	28.4	
7	CL-Cl	23	25	2	279.87	279.87	279.87	0	0.0044	0	4.2	7.4	9.8	11.6	14.2	
8	Cl-CH	25	26.5	1.5	293.29	293.29	293.29	0	0.005	0	3.6	6.3	8.4	9.9	12.1	
9	Cl-CH	26.5	28.5	2	305.79	305.79	305.79	0	0.005	0	4.8	8.5	11.1	13.2	16.1	
10	Cl-CH	28.5	32.5	4	327.34	327.34	327.34	0	0.005	0	9.5	16.9	22.3	26.4	32.3	
11	Cl-CH	32.5	36.5	4	356.3	356.3	356.3	0	0.005	0	9.5	16.9	22.3	26.4	32.3	
12	Cl-CH	36.5	38.5	2	378.18	378.18	378.18	0	0.005	0	4.8	8.5	11.1	13.2	16.1	
13	Cl-CH	38.5	40	1.5	391	391	391	0	0.005	0	3.6	6.3	8.4	9.9	12.1	
14	CL-ML	40	42	2	405.35	405.35	405.35	0	0.0036	0	3.4	6.1	8.0	9.5	11.6	
15	CL-ML	42	45	3	427.53	427.53	427.53	0	0.0036	0	5.2	9.1	12.0	14.3	17.4	
16	CL-ML	45	47	2	449.82	449.82	449.82	0	0.0036	0	3.4	6.1	8.0	9.5	11.6	
											76.7	135.9	179.1	212.6	259.3	

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: SBL North Abutment - Geofoam - Case E to E- O/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	Vertical Eff. Stress (kPa)		Rs' ( $\sigma_v/\sigma_v'-1$ )	Cas	Settlement due to Secondary Consolidation (mm)									
		From	To		$\sigma_v'$ with surcharge	$\sigma_v'$ w/o surcharge			1	3	6	10	20	Time after end of Primary Consolidation (Years)				
1	Clayey Silt	5	6.5	1.5	184.16	130.66	0.40945967	0.0026	0.0	0.0	0.0	0.0	0.0					
2	Clayey Silt	6.5	8	1.5	211.72	143.47	0.47570921	0.0026	0.0	0.0	0.0	0.0	0.0					
3	SP&ML (NP)	8	13	5	170.96	170.96	0	0	0.0	0.0	0.0	0.0	0.0					
4	CL-Cl	13	15	2	199.92	199.92	0	0.0044	4.2	7.4	9.8	11.6	14.2					
5	CL-Cl	15	19	4	223.71	223.71	0	0.0044	8.4	14.9	19.6	23.3	28.4					
6	CL-Cl	19	23	4	255.66	255.66	0	0.0044	8.4	14.9	19.6	23.3	28.4					
7	CL-Cl	23	25	2	279.82	279.82	0	0.0044	4.2	7.4	9.8	11.6	14.2					
8	CL-CH	25	26.5	1.5	295.63	293.26	0.00808157	0.005	0.0	0.0	0.0	0.0	0.0					
9	CL-CH	26.5	28.5	2	310.43	305.75	0.01530662	0.005	0.0	0.0	0.0	0.0	0.0					
10	CL-CH	28.5	32.5	4	335.81	327.31	0.02596926	0.005	0.0	0.0	0.0	0.0	0.0					
11	CL-CH	32.5	36.5	4	369.64	356.28	0.0374986	0.005	0.0	0.0	0.0	0.0	0.0					
12	CL-CH	36.5	38.5	2	395.02	378.17	0.04455668	0.005	0.0	0.0	0.0	0.0	0.0					
13	CL-CH	38.5	40	1.5	409.83	390.99	0.04818538	0.005	0.0	0.0	0.0	0.0	0.0					
14	CL-ML	40	42	2	406.91	405.35	0.00384853	0.0036	0.0	0.0	0.0	0.0	0.0					
15	CL-ML	42	45	3	431.03	427.53	0.00818656	0.0036	0.0	0.0	0.0	0.0	0.0					
16	CL-ML	45	47	2	455.15	449.78	0.01193917	0.0036	0.0	0.0	0.0	0.0	0.0					
greater of pre-consolidation pressure or $\sigma_v'$									25.2	44.6	58.8	69.8	85.2					

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: SBL North Approach-1 - [Rockfill + Surcharge] to Rockfill - Case A to Case B - N/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs <sup>1</sup> (σ <sub>v</sub> /σ <sub>v</sub> )-1	C <sub>as</sub>	Time after end of Primary Consolidation (years)					
		From	To		Vertical Eff. Stress (kPa)		σ <sub>v</sub>				Settlement due to Secondary Consolidation (mm)					
					σ <sub>v</sub> with surcharge	σ <sub>v</sub> w/o surcharge	σ <sub>v</sub>	σ <sub>v</sub>								
1	Clayey Silt	8	10	2	242.21	199.19	213.46	213.46	0.2159747	0.0026	0.3	1.1	1.8	2.5	3.6	
2	Clayey Silt	10	12	2	250.21	213.46	228.27	213.46	0.1721634	0.0026	0.7	1.9	2.8	3.6	4.9	
3	SP&ML (NP)	12	14	2	269.65	228.27	273.06	228.27	0.18127656	0	0.0	0.0	0.0	0.0	0.0	
4	CL-Cl	14	15.5	1.5	273.06	240.6	282.13	240.6	0.13491272	0.0044	1.9	3.8	5.2	6.4	8.1	
5	CL-Cl	15.5	19	3.5	282.13	257.37	307.24	257.37	0.09620391	0.0044	7.5	13.0	16.9	19.9	24.2	
6	CL-Cl	19	22.5	3.5	307.24	281.81	323.43	281.81	0.0902381	0.0044	8.1	13.8	17.7	20.8	25.2	
7	CL-Cl	22.5	24	1.5	323.43	299.68	332.45	299.68	0.0792512	0.0044	4.0	6.5	8.3	9.6	11.5	
8	CL-CH	24	25.5	1.5	332.45	309.84	342.45	309.84	0.07297315	0.005	4.9	7.9	9.9	11.4	13.6	
9	CL-CH	25.5	27.5	2	342.45	321.01	358.72	321.01	0.0667892	0.005	7.1	11.1	13.9	16.0	18.9	
10	CL-CH	27.5	31	3.5	358.72	338.9	380.16	338.9	0.05848333	0.005	13.8	21.1	26.0	29.7	34.9	
11	CL-CH	31	34.5	3.5	380.16	362.15	402.33	362.15	0.04973077	0.005	15.5	23.0	27.9	31.7	37.0	
12	CL-CH	34.5	38	3.5	402.33	385.88	420.07	385.88	0.04262983	0.005	17.0	24.7	29.7	33.5	38.8	
13	CL-CH	38	40	2	420.07	404.81	431.41	404.81	0.0376967	0.005	10.4	14.9	17.8	20.0	23.0	
14	CL-CH	40	41.5	1.5	431.41	416.98	443.77	416.98	0.03460598	0.005	8.2	11.5	13.7	15.4	17.7	
15	CL-ML	41.5	43.5	2	443.77	430.75	448.16	430.75	0.03022635	0.0036	8.4	11.6	13.8	15.4	17.6	
16	CL-ML	43.5	45.5	2	462.13	448.16		448.16	0.0311719	0.0036	8.2	11.5	13.6	15.2	17.4	
											116.1	177.5	219.0	251.1	296.4	



**Job Number:** 15-64-15  
**Job Description:** Hwy69 - Four-Laning - Estaire  
**Case:** SBL North Approach-1 - [Rockfill + Surcharge] to Rockfill - Case A to Case B - O/C

**Duration of Primary Consolidation (tp) (years):** 0.5      years       $\longrightarrow$       6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs' (σvs/σ'v)-1	Cαs	Time after end of Primary Consolidation (years)						
		From	To		Vertical Eff. Stress (kPa)		σ'v	σ'vs			w/o surcharge	Settlement due to Secondary Consolidation (mm)					
					σ'vs	w/o surcharge						1	3	6	10	20	
1	Clayey Silt	8	10	2	241.28	198.59	0.21496551	0.0026	0.3	1.1	1.8	2.5	3.6				
2	Clayey Silt	10	12	2	250.14	213.14	0.17359482	0.0026	0.7	1.8	2.8	3.6	4.8				
3	SP&ML (NP)	12	14	2	268.31	227.75	0.17809001	0	0.0	0.0	0.0	0.0	0.0				
4	CL-Cl	14	15.5	1.5	272.36	240.11	0.13431344	0.0044	1.9	3.8	5.2	6.4	8.1				
5	CL-Cl	15.5	19	3.5	281.96	257.18	0.09635275	0.0044	7.5	13.0	16.9	19.9	24.2				
6	CL-Cl	19	22.5	3.5	307.03	281.7	0.08991835	0.0044	8.2	13.8	17.8	20.8	25.2				
7	CL-Cl	22.5	24	1.5	323.41	299.61	0.0794366	0.0044	4.0	6.5	8.3	9.6	11.5				
8	CL-CH	24	25.5	1.5	332.48	309.8	0.07320852	0.005	4.9	7.9	9.9	11.4	13.6				
9	CL-CH	25.5	27.5	2	342.53	320.99	0.06710489	0.005	7.1	11.1	13.8	15.9	18.9				
10	CL-CH	27.5	31	3.5	358.85	338.92	0.05880444	0.005	13.7	21.0	25.9	29.6	34.8				
11	CL-CH	31	34.5	3.5	380.37	362.21	0.05013666	0.005	15.4	22.9	27.8	31.6	36.9				
12	CL-CH	34.5	38	3.5	402.59	385.98	0.04303332	0.005	16.9	24.6	29.6	33.4	38.7				
13	CL-CH	38	40	2	420.32	404.93	0.03800657	0.005	10.4	14.8	17.7	19.9	23.0				
14	CL-CH	40	41.5	1.5	431.69	417.12	0.03493	0.005	8.1	11.5	13.7	15.3	17.6				
15	CL-ML	41.5	43.5	2	444.24	430.9	0.03095846	0.0036	8.3	11.5	13.7	15.3	17.5				
16	CL-ML	43.5	45.5	2	462.04	448.08	0.03115515	0.0036	8.3	11.5	13.6	15.2	17.4				
									115.7	177.0	218.5	250.5	295.9				

**Job Number:** 15-64-15  
**Job Description:** Hwy69 - Four-Laning - Estaire  
**Case:** SBL North Approach-1 - [Sandfill + Surcharge] to Sandfill - Case C to Case D - N/C

**Duration of Primary Consolidation (tp) (years):** 0.5      years       $\longrightarrow$       6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs' ( $\sigma'_{vs}/\sigma'_{vf}$ )-1	Cas	Time after end of Primary Consolidation (years)					
		From	To		Vertical Eff. Stress (kPa)		σ <sub>vf</sub>				Settlement due to Secondary Consolidation (mm)					
					σ <sub>vs</sub> with surcharge	w/o surcharge	σ <sub>vs</sub>	σ <sub>vf</sub>								
1	Clayey Silt	8	10	2	260.03	216.89	0.19890267	0.0026	0.3	1.2	2.0	2.7	3.9			
2	Clayey Silt	10	12	2	267.3	230.97	0.15729315	0.0026	1.0	2.3	3.3	4.2	5.5			
3	SP&ML (NP)	12	14	2	288.56	245.63	0.17477507	0	0.0	0.0	0.0	0.0	0.0			
4	CL-Cl	14	15.5	1.5	290.11	257.69	0.12581008	0.0044	2.2	4.2	5.7	6.8	8.6			
5	CL-Cl	15.5	19	3.5	297.35	273.87	0.08573411	0.0044	8.6	14.4	18.4	21.5	25.9			
6	CL-Cl	19	22.5	3.5	322.74	297.48	0.08491327	0.0044	8.7	14.5	18.5	21.6	26.0			
7	CL-Cl	22.5	24	1.5	338.45	314.72	0.07540036	0.0044	4.2	6.8	8.5	9.9	11.8			
8	Cl-CH	24	25.5	1.5	347.08	324.49	0.06961694	0.005	5.1	8.1	10.2	11.7	13.9			
9	Cl-CH	25.5	27.5	2	356.61	335.19	0.06390405	0.005	7.4	11.5	14.2	16.3	19.3			
10	Cl-CH	27.5	31	3.5	372.16	352.37	0.05616256	0.005	14.2	21.6	26.5	30.2	35.4			
11	Cl-CH	31	34.5	3.5	392.74	374.77	0.04794941	0.005	15.8	23.4	28.4	32.2	37.4			
12	Cl-CH	34.5	38	3.5	414.11	397.71	0.04123608	0.005	17.3	25.1	30.1	33.9	39.2			
13	Cl-CH	38	40	2	431.22	416.05	0.03646196	0.005	10.6	15.1	18.0	20.2	23.2			
14	Cl-CH	40	41.5	1.5	442.11	427.88	0.03325699	0.005	8.3	11.7	13.9	15.6	17.8			
15	CL-ML	41.5	43.5	2	453.82	441.31	0.02834742	0.0036	8.6	11.9	14.0	15.6	17.8			
16	CL-ML	43.5	45.5	2	472.41	458.44	0.03047291	0.0036	8.3	11.6	13.7	15.3	17.5			
									120.8	183.3	225.3	257.7	303.3			

**Job Number:** 15-64-15  
**Job Description:** Hwy69 - Four-Laning - Estaire  
**Case:** SBL North Approach-1 - [Sandfill + Surcharge] to Sandfill - Case C to Case D - O/C

**Duration of Primary Consolidation (tp) (years):** 0.5 years  $\longrightarrow$  6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs' (σvs/σ'v) <sup>-1</sup>	Cuc	Time after end of Primary Consolidation (years)					
		From	To		Vertical Eff. Stress (kPa)		Settlement due to Secondary Consolidation (mm)									
					σ'vs with surcharge	σ'v w/o surcharge	1	3			6	10	20			
1	Clayey Silt	8	10	2	258.84	215.95	0.19861079	0.0026	0.3	1.2	2.0	2.7	3.9			
2	Clayey Silt	10	12	2	267.06	230.36	0.15931585	0.0026	1.0	2.2	3.2	4.1	5.4			
3	SP&ML (NP)	12	14	2	286.83	244.78	0.17178691	0	0.0	0.0	0.0	0.0	0.0			
4	CL-Cl	14	15.5	1.5	289.17	256.89	0.1256569	0.0044	2.2	4.2	5.7	6.9	8.6			
5	CL-Cl	15.5	19	3.5	296.96	273.44	0.08601521	0.0044	8.6	14.3	18.3	21.4	25.8			
6	CL-Cl	19	22.5	3.5	322.29	297.16	0.08456724	0.0044	8.7	14.5	18.5	21.6	26.1			
7	CL-Cl	22.5	24	1.5	338.22	314.46	0.0755581	0.0044	4.2	6.8	8.5	9.9	11.8			
8	CL-CH	24	25.5	1.5	346.91	324.27	0.06981836	0.005	5.1	8.1	10.2	11.7	13.9			
9	CL-CH	25.5	27.5	2	356.5	335.01	0.06414734	0.005	7.3	11.4	14.2	16.3	19.2			
10	CL-CH	27.5	31	3.5	372.13	352.25	0.05643719	0.005	14.2	21.5	26.4	30.1	35.4			
11	CL-CH	31	34.5	3.5	392.8	374.7	0.04830531	0.005	15.8	23.3	28.3	32.1	37.3			
12	CL-CH	34.5	38	3.5	414.23	397.7	0.04156399	0.005	17.3	25.0	30.0	33.8	39.1			
13	CL-CH	38	40	2	431.32	416.08	0.03662757	0.005	10.6	15.0	18.0	20.1	23.2			
14	CL-CH	40	41.5	1.5	442.25	427.93	0.03346342	0.005	8.3	11.7	13.9	15.5	17.8			
15	CL-ML	41.5	43.5	2	454.24	441.38	0.02913589	0.0036	8.5	11.8	13.9	15.5	17.7			
16	CL-ML	43.5	45.5	2	472.19	458.23	0.03046505	0.0036	8.3	11.6	13.7	15.3	17.5			
									120.4	182.8	224.8	257.1	302.8			

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: SBL North Approach-1 - Sandfill to Geofoam - Case D to Case E - N/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=6m)		(H=6m)		Rs ( $\sigma_v/\sigma'v$ )-1	Cur	Time after end of Primary Consolidation (years)					
					Vertical Eff. Stress (kPa)		Settlement due to Secondary Consolidation (mm)									
					From	To	$\sigma_v$ with surcharge	$\sigma'v$ w/o surcharge			1	3	6	10	20	
		1	Clayey Silt	8	10	2	216.89	154.13	0.40718874	0.0026	0.0	0.4	0.8	1.2	1.8	
2	Clayey Silt	10	12	2	230.97	171	0.35070175	0.0026	0.1	0.5	1.0	1.4	2.2			
3	SP&ML (NP)	12	14	2	245.63	187.9	0.30723789	0	0.0	0.0	0.0	0.0	0.0			
4	CL-Cl	14	15.5	1.5	257.69	202.29	0.27386425	0.0044	0.2	0.9	1.7	2.4	3.6			
5	CL-Cl	15.5	19	3.5	273.87	222.19	0.23259373	0.0044	0.7	2.8	4.9	6.8	9.9			
6	CL-Cl	19	22.5	3.5	297.48	250.28	0.18858878	0.0044	1.5	4.3	6.8	9.1	12.6			
7	CL-Cl	22.5	24	1.5	314.72	270.5	0.16347505	0.0044	1.1	2.7	3.9	5.0	6.6			
8	Cl-CH	24	25.5	1.5	324.49	281.95	0.15087782	0.005	1.6	3.6	5.1	6.3	8.2			
9	Cl-CH	25.5	27.5	2	335.19	294.5	0.13816638	0.005	2.7	5.5	7.7	9.4	12.0			
10	Cl-CH	27.5	31	3.5	352.37	314.33	0.12101931	0.005	6.1	11.6	15.6	18.8	23.5			
11	Cl-CH	31	34.5	3.5	374.77	339.77	0.10301086	0.005	7.8	13.9	18.2	21.5	26.4			
12	Cl-CH	34.5	38	3.5	397.71	365.39	0.08845343	0.005	9.5	15.9	20.4	23.9	28.9			
13	Cl-CH	38	40	2	416.05	385.63	0.0788839	0.005	6.1	10.0	12.6	14.6	17.5			
14	Cl-CH	40	41.5	1.5	427.88	398.56	0.07356483	0.005	4.9	7.8	9.9	11.4	13.6			
15	CL-ML	41.5	43.5	2	441.31	413.02	0.06849547	0.0036	5.0	7.9	9.8	11.3	13.5			
16	CL-ML	43.5	45.5	2	458.44	430.89	0.06393743	0.0036	5.3	8.2	10.2	11.7	13.9			
									52.7	96.1	128.6	155.0	194.2			

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: SBL North Approach-1 - Sandfill to Geofoam - Case D to Case E - O/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=6m)		Rs' ( $\sigma_{vs}/\sigma'_{vf}$ )-1	Gas	Time after end of Primary Consolidation (years)									
		From	To		$\sigma_{vs}$ with surcharge	Vertical Eff. Stress (kPa) $\sigma'_{vf}$ w/o surcharge			1	2	3	6	10	20	Settlement due to Secondary Consolidation (mm)			
1	Clayey Silt	8	10	2	215.95	154.04	0.4019086	0.0026	0.0	0.4	0.8	1.0	1.2	1.9				
2	Clayey Silt	10	12	2	230.36	170.95	0.34752852	0.0026	0.1	0.5	1.0	1.0	1.4	2.2				
3	SP&ML (NP)	12	14	2	244.78	187.82	0.30326909	0	0.0	0.0	0.0	0.0	0.0	0.0				
4	CL-Cl	14	15.5	1.5	256.89	202.22	0.27034912	0.0044	0.2	1.0	1.7	2.5	2.5	3.6				
5	CL-Cl	15.5	19	3.5	273.44	222.15	0.23088004	0.0044	0.7	2.9	5.0	6.9	8.9	10.0				
6	CL-Cl	19	22.5	3.5	297.16	250.25	0.18745255	0.0044	1.5	4.4	6.9	9.2	9.2	12.7				
7	CL-Cl	22.5	24	1.5	314.46	270.48	0.16259982	0.0044	1.2	2.7	4.0	5.0	5.0	6.7				
8	CL-CH	24	25.5	1.5	324.27	281.93	0.15017912	0.005	1.7	3.6	5.1	6.4	6.4	8.3				
9	CL-CH	25.5	27.5	2	335.01	294.48	0.13763244	0.005	2.7	5.6	7.7	9.4	9.4	12.0				
10	CL-CH	27.5	31	3.5	352.25	314.32	0.1206732	0.005	6.2	11.7	15.7	18.2	18.9	23.6				
11	CL-CH	31	34.5	3.5	374.7	339.76	0.1028373	0.005	7.9	13.9	18.2	21.6	21.6	26.4				
12	CL-CH	34.5	38	3.5	397.7	365.38	0.08845585	0.005	9.5	15.9	20.4	23.9	23.9	28.9				
13	CL-CH	38	40	2	416.08	385.63	0.0789617	0.005	6.1	10.0	12.6	14.6	14.6	17.5				
14	CL-CH	40	41.5	1.5	427.93	398.56	0.07369029	0.005	4.9	7.8	9.8	11.4	11.4	13.6				
15	CL-ML	41.5	43.5	2	441.38	413.02	0.06866496	0.0036	5.0	7.9	9.8	11.3	11.3	13.5				
16	CL-ML	43.5	45.5	2	458.23	430.87	0.06349943	0.0036	5.3	8.3	10.3	11.8	11.8	13.9				
									52.8	96.4	129.0	155.4	155.4	194.7				

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: SBL North Approach-1 - Geofoam - Case E to Case E - N/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	Vertical Eff. Stress (kPa)		R <sub>s</sub> (σ <sub>vs</sub> /σ <sub>v'</sub> )-1	C <sub>as</sub>	Time at end of Primary Consolidation (years)					
		From	To		σ <sub>vs</sub> with surcharge	σ <sub>v'</sub> w/o surcharge			1	3	6	10	15	20
1	Clayey Silt	8	10	2	154.13	154.13	0	0.0026	0.0	0.4	0.8	1.2	1.8	
2	Clayey Silt	10	12	2	171	171	0	0.0026	0.1	0.5	1.0	1.4	2.2	
3	SP&ML (NP)	12	14	2	187.9	187.9	0	0	0.0	0.0	0.0	0.0	0.0	
4	CL-Cl	14	15.5	1.5	202.29	202.29	0	0.0044	0.2	0.9	1.7	2.4	3.6	
5	CL-Cl	15.5	19	3.5	222.19	222.19	0	0.0044	0.7	2.8	4.9	6.8	9.9	
6	CL-Cl	19	22.5	3.5	250.28	250.28	0	0.0044	1.5	4.3	6.8	9.1	12.6	
7	CL-Cl	22.5	24	1.5	270.5	270.5	0	0.0044	1.1	2.7	3.9	5.0	6.6	
8	CL-CH	24	25.5	1.5	281.95	281.95	0	0.005	1.6	3.6	5.1	6.3	8.2	
9	CL-CH	25.5	27.5	2	294.5	294.5	0	0.005	2.7	5.5	7.7	9.4	12.0	
10	CL-CH	27.5	31	3.5	314.33	314.33	0	0.005	6.1	11.6	15.6	18.8	23.5	
11	CL-CH	31	34.5	3.5	339.77	339.77	0	0.005	7.8	13.9	18.2	21.5	26.4	
12	CL-CH	34.5	38	3.5	365.39	365.39	0	0.005	9.5	15.9	20.4	23.9	28.9	
13	CL-CH	38	40	2	385.63	385.63	0	0.005	6.1	10.0	12.6	14.6	17.5	
14	CL-CH	40	41.5	1.5	398.56	398.56	0	0.005	4.9	7.8	9.9	11.4	13.6	
15	CL-ML	41.5	43.5	2	413.02	413.02	0	0.0036	5.0	7.9	9.8	11.3	13.5	
16	CL-ML	43.5	45.5	2	430.89	430.89	0	0.0036	5.3	8.2	10.2	11.7	13.9	
									52.7	96.1	128.6	155.0	194.2	

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: SBL North Approach-1 - Geofoam - Case E to Case E - O/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	Vertical Eff. Stress (kPa)		Rs' (σvs/σ'v) <sup>-1</sup>	C <sub>α</sub>	Time after end of Primary Consolidation (years)									
		From	To		σ'vs				σ'v	Settlement due to Secondary Consolidation (mm)	1	2	3	6	10	20		
					with surcharge	w/o surcharge												
1	Clayey Silt	8	10	2	182.93	154.04	0.18754869	0.0026	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
2	Clayey Silt	10	12	2	210.5	170.95	0.2313542	0.0026	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
3	SP&ML (NP)	12	14	2	187.82	187.82	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
4	CL-Cl	14	15.5	1.5	202.22	202.22	0	0.0044	3.1	5.6	7.4	8.7	10.6	12.5	14.4	16.3		
5	CL-Cl	15.5	19	3.5	222.15	222.15	0	0.0044	7.3	13.0	17.2	20.4	24.8	29.2	33.6	38.0		
6	CL-Cl	19	22.5	3.5	250.25	250.25	0	0.0044	7.3	13.0	17.2	20.4	24.8	29.2	33.6	38.0		
7	CL-Cl	22.5	24	1.5	270.48	270.48	0	0.0044	3.1	5.6	7.4	8.7	10.6	12.5	14.4	16.3		
8	CL-CH	24	25.5	1.5	286.62	281.93	0.01663534	0.005	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
9	CL-CH	25.5	27.5	2	301.42	294.48	0.02356697	0.005	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
10	CL-CH	27.5	31	3.5	324.69	314.32	0.03299186	0.005	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
11	CL-CH	31	34.5	3.5	354.29	339.76	0.04276548	0.005	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
12	CL-CH	34.5	38	3.5	383.9	365.38	0.05068696	0.005	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
13	CL-CH	38	40	2	407.17	385.63	0.05858665	0.005	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
14	CL-CH	40	41.5	1.5	421.96	398.56	0.05871136	0.005	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
15	CL-ML	41.5	43.5	2	418.49	413.02	0.01324391	0.0036	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
16	CL-ML	43.5	45.5	2	437.79	430.87	0.01606053	0.0036	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
									21.0	37.2	49.0	58.2	67.4	76.6	85.8	95.0		

↑ greater of pre-consolidation pressure or σ<sub>v</sub>'s

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: SBL North Approach- 2 - [Rockfill + Surcharge] to Rockfill - Case A to Case B - N/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs' (σ <sub>v</sub> s/σ <sub>v</sub> h)-1	Cas	Time after end of Primary Consolidation (years)						
		From	To		σ <sub>v</sub> 's with surcharge	Vertical Eff. Stress (kPa) σ <sub>v</sub> ' w/o surcharge	1	3			6	10	15	20			
Settlement due to Secondary Consolidation (mm)																	
1	CL-ML	4	5.5	1.5	228.91	176.18	0.29929617	0.0036	0.1	0.7	1.2	1.8	2.6				
2	CL-ML	5.5	7	1.5	234.29	187.34	0.25061386	0.0036	0.2	0.9	1.6	2.2	3.2				
3	SP&ML (NP)	7	10.5	3.5	256.91	206.21	0.24586586	0	0.0	0.0	0.0	0.0	0.0				
4	CL-CI	10.5	12.5	2	263.12	225.32	0.16776141	0.0044	1.4	3.4	5.0	6.4	8.5				
5	CL-CI	12.5	16.25	3.75	271.36	243.61	0.11391158	0.0044	6.4	11.8	15.6	18.7	23.2				
6	CL-CI	16.25	20	3.75	296.23	268.47	0.10340075	0.0044	7.4	13.0	17.1	20.2	24.8				
7	CL-CI	20	22	2	313.28	288.05	0.08758896	0.0044	4.8	8.1	10.3	12.1	14.6				
8	CI-CH	22	23.5	1.5	323.2	299.5	0.07913189	0.005	4.6	7.5	9.4	11.0	13.1				
9	CI-CH	23.5	25.5	2	332.56	310.17	0.07218622	0.005	6.6	10.6	13.3	15.3	18.3				
10	CI-CH	25.5	28.5	3	346.48	325.76	0.06360511	0.005	11.1	17.2	21.3	24.5	28.9				
11	CI-CH	28.5	31.5	3	363.9	344.92	0.05502725	0.005	12.4	18.7	22.9	26.1	30.6				
12	CI-CH	31.5	34.5	3	381.99	364.55	0.0478398	0.005	13.6	20.1	24.3	27.6	32.1				
13	CI-CH	34.5	36.5	2	397.44	381.2	0.04260231	0.005	9.7	14.1	17.0	19.2	22.2				
14	CI-CH	36.5	38	1.5	408.4	393.01	0.03915931	0.005	7.6	11.0	13.2	14.8	17.1				
15	CL-ML	38	40.5	2.5	422.04	408.54	0.0330445	0.004	11.1	15.7	18.6	20.8	23.8				
16	CL-ML	40.5	43	2.5	444.63	429.98	0.03407135	0.004	11.0	15.5	18.4	20.6	23.6				
									108.0	168.1	209.3	241.3	286.9				



**Job Number:** 15-64-15  
**Job Description:** Hwy69 - Four-Laning - Estaire  
**Case:** SBL North Approach- 2 - [Rockfill + Surcharge] to Rockfill - Case A to Case B - O/C

**Duration of Primary Consolidation (tp) (years):** 0.5      years       $\longrightarrow$       6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs' ( $\sigma'_{vs}/\sigma'_{vf}$ )-1	C <sub>as</sub>	Time after end of Primary Consolidation (years)					
		From	To		σ'vs with surcharge	Vertical Eff. Stress (kPa)	σ'vf w/o surcharge	Settlement due to Secondary Consolidation (mm)								
								1			3	6	10	20		
1	CL-ML	4	5.5	1.5	228.1	175.34	0.30090111	0.0036	0.1	0.7	1.2	1.8	2.6			
2	CL-ML	5.5	7	1.5	233.78	186.81	0.25143194	0.0036	0.2	0.9	1.6	2.2	3.2			
3	SP&ML (NP)	7	10.5	3.5	255.97	205.42	0.2460812	0	0.0	0.0	0.0	0.0	0.0			
4	CL-Cl	10.5	12.5	2	262.41	224.6	0.16834372	0.0044	1.4	3.4	5.0	6.4	8.5			
5	CL-Cl	12.5	16.25	3.75	271.02	243.27	0.11407079	0.0044	6.4	11.7	15.6	18.7	23.2			
6	CL-Cl	16.25	20	3.75	295.92	268.24	0.10319117	0.0044	7.4	13.1	17.1	20.3	24.9			
7	CL-Cl	20	22	2	313.13	287.9	0.0876346	0.0044	4.8	8.1	10.3	12.1	14.6			
8	Cl-CH	22	23.5	1.5	323.1	299.38	0.07923041	0.005	4.6	7.4	9.4	10.9	13.1			
9	Cl-CH	23.5	25.5	2	332.5	310.09	0.07226934	0.005	6.6	10.6	13.3	15.3	18.3			
10	Cl-CH	25.5	28.5	3	346.48	325.73	0.06370307	0.005	11.1	17.2	21.3	24.5	28.9			
11	Cl-CH	28.5	31.5	3	363.95	344.95	0.05508045	0.005	12.4	18.7	22.9	26.1	30.6			
12	Cl-CH	31.5	34.5	3	382.09	364.62	0.0479129	0.005	13.6	20.1	24.3	27.6	32.1			
13	Cl-CH	34.5	36.5	2	397.54	381.3	0.04259114	0.005	9.7	14.1	17.0	19.2	22.2			
14	Cl-CH	36.5	38	1.5	408.51	393.14	0.03909549	0.005	7.7	11.0	13.2	14.8	17.1			
15	CL-ML	38	40.5	2.5	422.31	408.68	0.03335128	0.004	11.1	15.6	18.5	20.7	23.8			
16	CL-ML	40.5	43	2.5	444.49	429.84	0.03408245	0.004	11.0	15.5	18.4	20.6	23.6			
									107.9	168.0	209.1	241.1	286.7			

**Job Number:** 15-64-15  
**Job Description:** Hwy69 - Four-Laning - Estaire  
**Case:** SBL North Approach- 2 - [Sandfill + Surcharge] to Sandfill - Case C to Case D - N/C

**Duration of Primary Consolidation (tp) (years):** 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs' (σ <sub>v</sub> /σ <sub>vH</sub> ) <sup>-1</sup>	C <sub>as</sub>	Time after end of Primary Consolidation (years)						
		From	To		Vertical Eff. Stress (kPa)		σ <sub>v</sub> with surcharge	σ <sub>v</sub> ' w/o surcharge			1	3	6	10	20		
					σ <sub>v</sub>	σ <sub>v</sub> '											
Settlement due to Secondary Consolidation (mm)																	
1	CL-ML	4	5.5	1.5	249.09	196.13	196.13	0.27002498	0.0036	0.0036	0.2	0.8	1.4	2.0	3.0		
2	CL-ML	5.5	7	1.5	254.13	207.33	207.33	0.2257271	0.0036	0	0.3	1.0	1.8	2.5	3.6		
3	SP&ML (NP)	7	10.5	3.5	279.14	226.46	226.46	0.23262386	0	0	0.0	0.0	0.0	0.0	0.0		
4	CL-CI	10.5	12.5	2	283.59	245.57	245.57	0.15482347	0.0044	0.0044	1.8	4.0	5.7	7.2	9.4		
5	CL-CI	12.5	16.25	3.75	289.96	263.48	263.48	0.10050099	0.0044	0.0044	7.6	13.4	17.5	20.7	25.3		
6	CL-CI	16.25	20	3.75	315.29	287.69	287.69	0.0959366	0.0044	0.0044	8.1	14.0	18.1	21.4	26.0		
7	CL-CI	20	22	2	331.8	306.61	306.61	0.08215649	0.0044	0.0044	5.2	8.5	10.8	12.6	15.1		
8	CI-CH	22	23.5	1.5	341.26	317.6	317.6	0.07449622	0.005	0.005	4.8	7.8	9.8	11.3	13.5		
9	CI-CH	23.5	25.5	2	350.12	327.77	327.77	0.06818806	0.005	0.005	7.0	11.0	13.7	15.8	18.7		
10	CI-CH	25.5	28.5	3	363.32	342.65	342.65	0.06032395	0.005	0.005	11.6	17.8	21.9	25.1	29.6		
11	CI-CH	28.5	31.5	3	379.88	360.96	360.96	0.05241578	0.005	0.005	12.8	19.2	23.4	26.6	31.1		
12	CI-CH	31.5	34.5	3	397.14	379.77	379.77	0.04573821	0.005	0.005	14.0	20.5	24.8	28.0	32.6		
13	CI-CH	34.5	36.5	2	411.89	395.77	395.77	0.04073073	0.005	0.005	10.0	14.4	17.3	19.5	22.5		
14	CI-CH	36.5	38	1.5	422.33	407.14	407.14	0.03730903	0.005	0.005	7.9	11.2	13.4	15.0	17.3		
15	CL-ML	38	40.5	2.5	434.97	422.18	422.18	0.03029513	0.004	0.004	11.6	16.2	19.1	21.3	24.4		
16	CL-ML	40.5	43	2.5	457.8	443.15	443.15	0.0305878	0.004	0.004	11.1	15.7	18.6	20.8	23.8		
											113.8	175.4	217.3	249.8	295.9		

**Job Number:** 15-64-15  
**Job Description:** Hwy69 - Four-Laning - Estaire  
**Case:** SBL North Approach- 2 - [Sandfill + Surcharge] to Sandfill - Case C to Case D - O/C

**Duration of Primary Consolidation (tp) (years):** 0.5 years  $\longrightarrow$  6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs' (σ <sub>vs</sub> /σ <sub>vf</sub> )-1	C <sub>αs</sub>	Time after end of Primary Consolidation (years)					
		From	To		Vertical Eff. Stress (kPa)		Settlement due to Secondary Consolidation (mm)									
					σ <sub>vs</sub> with surcharge	σ <sub>vf</sub> w/o surcharge	1	3			6	10	20			
1	CL-ML	4	5.5	1.5	248.04	194.98	0.27213047	0.0036	0.2	0.8	1.4	2.0	3.0			
2	CL-ML	5.5	7	1.5	253.32	206.51	0.22667183	0.0036	0.3	1.0	1.8	2.5	3.6			
3	SP&ML (NP)	7	10.5	3.5	278.06	225.36	0.23384807	0	0.0	0.0	0.0	0.0	0.0			
4	CL-Cl	10.5	12.5	2	282.64	244.55	0.15575547	0.0044	1.8	4.0	5.7	7.1	9.3			
5	CL-Cl	12.5	16.25	3.75	289.39	262.88	0.10084449	0.0044	7.6	13.4	17.4	20.6	25.2			
6	CL-Cl	16.25	20	3.75	314.73	287.21	0.09581839	0.0044	8.1	14.0	18.1	21.4	26.0			
7	CL-Cl	20	22	2	331.43	306.23	0.08229109	0.0044	5.1	8.5	10.8	12.6	15.1			
8	Cl-CH	22	23.5	1.5	340.94	317.27	0.07460523	0.005	4.8	7.8	9.8	11.3	13.5			
9	Cl-CH	23.5	25.5	2	349.86	327.49	0.06830743	0.005	7.0	11.0	13.7	15.8	18.7			
10	Cl-CH	25.5	28.5	3	363.12	342.43	0.06042111	0.005	11.5	17.8	21.9	25.1	29.5			
11	Cl-CH	28.5	31.5	3	379.74	360.81	0.05246529	0.005	12.8	19.2	23.4	26.6	31.1			
12	Cl-CH	31.5	34.5	3	397.06	379.67	0.04580293	0.005	14.0	20.5	24.8	28.0	32.6			
13	Cl-CH	34.5	36.5	2	411.81	395.71	0.04068636	0.005	10.0	14.4	17.3	19.5	22.5			
14	Cl-CH	36.5	38	1.5	422.27	407.11	0.03723809	0.005	7.9	11.2	13.4	15.0	17.3			
15	CL-ML	38	40.5	2.5	435.14	422.19	0.03067339	0.004	11.5	16.1	19.0	21.2	24.3			
16	CL-ML	40.5	43	2.5	457.51	442.86	0.03308043	0.004	11.1	15.6	18.6	20.8	23.8			
									113.7	175.2	217.1	249.5	295.6			

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: SBL North Approach- 2 - Sandfill to Geofoam - Case D to Case E - N/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=6m)		(H=6m)		Rs' ( $\sigma'_{vs}/\sigma'_{vf}$ )-1	Case	Time after end of Primary Consolidation (years)						
		From	To		Vertical Eff. Stress (kPa)		$\sigma'_{vs}$ with surcharge	$\sigma'_{vf}$ w/o surcharge			1	3	6	10	20		
Settlement due to Secondary Consolidation (mm)																	
1	CL-ML	4	5.5	1.5	196.13	119.49	0.64139259	0.0036	0.0	0.1	0.3	0.6	1.0				
2	CL-ML	5.5	7	1.5	207.33	132.36	0.56640979	0.0036	0.0	0.2	0.4	0.7	1.2				
3	SP&ML (NP)	7	10.5	3.5	226.46	153.69	0.47348559	0	0.0	0.0	0.0	0.0	0.0				
4	CL-CI	10.5	12.5	2	245.57	176.41	0.39204127	0.0044	0.1	0.7	1.4	2.1	3.2				
5	CL-CI	12.5	16.25	3.75	263.48	199.12	0.32322218	0.0044	0.3	1.8	3.4	4.9	7.5				
6	CL-CI	16.25	20	3.75	287.69	228.96	0.25650769	0.0044	0.6	2.6	4.7	6.6	9.6				
7	CL-CI	20	22	2	306.61	252	0.21670635	0.0044	0.5	1.8	3.0	4.2	6.0				
8	CI-CH	22	23.5	1.5	317.6	265.36	0.19686464	0.005	0.5	1.8	3.0	4.0	5.7				
9	CI-CH	23.5	25.5	2	327.77	277.8	0.17987761	0.005	1.2	3.2	4.9	6.5	8.8				
10	CI-CH	25.5	28.5	3	342.65	295.67	0.15889336	0.005	2.8	6.5	9.4	11.8	15.5				
11	CI-CH	28.5	31.5	3	360.96	317.28	0.1376702	0.005	4.1	8.3	11.5	14.2	18.1				
12	CI-CH	31.5	34.5	3	379.77	339.04	0.12013332	0.005	5.3	10.0	13.5	16.2	20.3				
13	CI-CH	34.5	36.5	2	395.77	357.28	0.10773063	0.005	4.2	7.6	10.0	11.9	14.6				
14	CI-CH	36.5	38	1.5	407.14	370.1	0.10008106	0.005	3.5	6.1	8.0	9.4	11.5				
15	CL-ML	38	40.5	2.5	422.18	386.68	0.09180718	0.004	5.2	8.8	11.4	13.3	16.2				
16	CL-ML	40.5	43	2.5	443.15	408.93	0.0836818	0.004	5.7	9.5	12.1	14.1	17.0				
									34.0	69.0	97.0	120.5	156.4				

**Job Number:** 15-64-15  
**Job Description:** Hwy69 - Four-Laning - Estaire  
**Case:** SBL North Approach- 2 - Sandfill to Geofoam - Case D to Case E - O/C

**Duration of Primary Consolidation (tp) (years):** 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=6m)		Rs' ( $\sigma'_{vs}/\sigma'_{vf}-1$ )	Gas	Time after end of Primary Consolidation (years)				
		From	To		Vertical Eff. Stress (kPa)				1	3	6	10	20
					$\sigma'_{vs}$ with surcharge	$\sigma'_{vf}$ w/o surcharge							
1	CL-ML	4	5.5	1.5	194.98	119.32	0.63409319	0.0036	0.0	0.1	0.3	0.6	1.0
2	CL-ML	5.5	7	1.5	206.51	132.25	0.56151229	0.0036	0.0	0.2	0.4	0.7	1.2
3	SP&ML (NP)	7	10.5	3.5	225.36	153.54	0.46776084	0	0.0	0.0	0.0	0.0	0.0
4	CL-CI	10.5	12.5	2	244.55	176.26	0.38743901	0.0044	0.1	0.7	1.4	2.1	3.3
5	CL-CI	12.5	16.25	3.75	262.88	199.04	0.32073955	0.0044	0.3	1.8	3.4	5.0	7.6
6	CL-CI	16.25	20	3.75	287.21	228.89	0.25479488	0.0044	0.6	2.6	4.7	6.6	9.7
7	CL-CI	20	22	2	306.23	251.94	0.21548781	0.0044	0.5	1.8	3.1	4.2	6.1
8	CL-CH	22	23.5	1.5	317.27	265.31	0.19584637	0.005	0.6	1.8	3.0	4.1	5.7
9	CL-CH	23.5	25.5	2	327.49	277.75	0.17908191	0.005	1.2	3.3	5.0	6.5	8.8
10	CL-CH	25.5	28.5	3	342.43	295.64	0.15826681	0.005	2.9	6.5	9.5	11.9	15.6
11	CL-CH	28.5	31.5	3	360.81	317.25	0.13730496	0.005	4.1	8.4	11.6	14.2	18.1
12	CL-CH	31.5	34.5	3	379.67	339.02	0.11990443	0.005	5.3	10.1	13.5	16.3	20.3
13	CL-CH	34.5	36.5	2	395.71	357.27	0.1075937	0.005	4.2	7.6	10.0	11.9	14.7
14	CL-CH	36.5	38	1.5	407.11	370.1	0.1	0.005	3.5	6.1	8.0	9.4	11.5
15	CL-ML	38	40.5	2.5	422.19	386.68	0.09183304	0.004	5.2	8.8	11.4	13.3	16.2
16	CL-ML	40.5	43	2.5	442.86	408.88	0.08310507	0.004	5.8	9.6	12.2	14.2	17.1
									34.2	69.4	97.4	121.0	156.9

**Job Number:** 15-64-15  
**Job Description:** Hwy69 - Four-Laning - Estaire  
**Case:** SBL North Approach- 2 - Geofoam - Case E to Case E - N/C

**Duration of Primary Consolidation (tp) (years):** 0.5      years       $\longrightarrow$       6 months

Layer	Soil	Depth (m)		Thickness (m)	Vertical Eff. Stress (kPa)		Rs' ( $\sigma'_v/\sigma'_{vf}-1$ )	C $\alpha$	Time after end of Primary Consolidation (years)				
		From	To		$\sigma'_v$ with surcharge	$\sigma'_{vf}$ w/o surcharge			1	3	6	10	20
1	CL-ML	4	5.5	1.5	119.49	119.49	0	0.0036	2.6	4.6	6.0	7.1	8.7
2	CL-ML	5.5	7	1.5	132.36	132.36	0	0.0036	2.6	4.6	6.0	7.1	8.7
3	SP&ML (NP)	7	10.5	3.5	153.69	153.69	0	0	0.0	0.0	0.0	0.0	0.0
4	CL-CI	10.5	12.5	2	176.41	176.41	0	0.0044	4.2	7.4	9.8	11.6	14.2
5	CL-CI	12.5	16.25	3.75	199.12	199.12	0	0.0044	7.9	13.9	18.4	21.8	26.6
6	CL-CI	16.25	20	3.75	228.96	228.96	0	0.0044	7.9	13.9	18.4	21.8	26.6
7	CI-CI	20	22	2	252	252	0	0.0044	4.2	7.4	9.8	11.6	14.2
8	CI-CH	22	23.5	1.5	265.36	265.36	0	0.005	3.6	6.3	8.4	9.9	12.1
9	CI-CH	23.5	25.5	2	277.8	277.8	0	0.005	4.8	8.5	11.1	13.2	16.1
10	CI-CH	25.5	28.5	3	295.67	295.67	0	0.005	7.2	12.7	16.7	19.8	24.2
11	CI-CH	28.5	31.5	3	317.28	317.28	0	0.005	7.2	12.7	16.7	19.8	24.2
12	CI-CH	31.5	34.5	3	339.04	339.04	0	0.005	7.2	12.7	16.7	19.8	24.2
13	CI-CH	34.5	36.5	2	357.28	357.28	0	0.005	4.8	8.5	11.1	13.2	16.1
14	CI-CH	36.5	38	1.5	370.1	370.1	0	0.005	3.6	6.3	8.4	9.9	12.1
15	CL-ML	38	40.5	2.5	386.68	386.68	0	0.004	4.8	8.5	11.1	13.2	16.1
16	CL-ML	40.5	43	2.5	408.93	408.93	0	0.004	4.8	8.5	11.1	13.2	16.1
									77.0	136.4	179.8	213.4	260.3

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: SBL North Approach- 2 - Geofoam - Case E to Case E - O/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	Vertical Eff. Stress (kPa)			Rs ( $\sigma'_{vs}/\sigma'_{vf}-1$ )	C <sub>as</sub>	Time after end of Primary Consolidation (Years)					
		From	To		σ' s With surcharge	σ' v w/o surcharge	Settlement due to Secondary Consolidation (mm)								
							1			3	6	10	20		
1	CL-ML	4	5.5	1.5	119.32	119.32	0	0.0036	2.6	4.6	6.0	7.1	8.7		
2	CL-ML	5.5	7	1.5	132.25	132.25	0	0.0036	2.6	4.6	6.0	7.1	8.7		
3	SP&ML (NP)	7	10.5	3.5	153.54	153.54	0	0	0.0	0.0	0.0	0.0	0.0		
4	CL-Cl	10.5	12.5	2	176.26	176.26	0	0.0044	4.2	7.4	9.8	11.6	14.2		
5	CL-Cl	12.5	16.25	3.75	199.04	199.04	0	0.0044	7.9	13.9	18.4	21.8	26.6		
6	CL-Cl	16.25	20	3.75	228.89	228.89	0	0.0044	7.9	13.9	18.4	21.8	26.6		
7	CL-Cl	20	22	2	251.94	251.94	0	0.0044	4.2	7.4	9.8	11.6	14.2		
8	Cl-CH	22	23.5	1.5	265.57	265.31	0.00097999	0.005	0.0	0.0	0.0	0.0	0.0		
9	Cl-CH	23.5	25.5	2	280.38	277.75	0.00946895	0.005	0.0	0.0	0.0	0.0	0.0		
10	Cl-CH	25.5	28.5	3	301.53	295.64	0.01992288	0.005	0.0	0.0	0.0	0.0	0.0		
11	Cl-CH	28.5	31.5	3	326.91	317.25	0.03044917	0.005	0.0	0.0	0.0	0.0	0.0		
12	Cl-CH	31.5	34.5	3	352.29	339.02	0.03914223	0.005	0.0	0.0	0.0	0.0	0.0		
13	Cl-CH	34.5	36.5	2	373.43	357.27	0.0452319	0.005	0.0	0.0	0.0	0.0	0.0		
14	Cl-CH	36.5	38	1.5	388.23	370.1	0.04898676	0.005	0.0	0.0	0.0	0.0	0.0		
15	CL-ML	38	40.5	2.5	388.7	386.68	0.00522396	0.004	0.0	0.0	0.0	0.0	0.0		
16	CL-ML	40.5	43	2.5	412.83	408.88	0.00966054	0.004	0.0	0.0	0.0	0.0	0.0		
									29.3	51.9	68.4	81.2	99.0		

greater of pre-consolidation pressure or  $\sigma'_{vs}$

**Job Number:** 15-64-15  
**Job Description:** Hwy69 - Four-Laning - Estaire  
**Case:** SBL North Approach- 3 - [Rockfill + Surcharge] to Rockfill - Case A to Case B - N/C

**Duration of Primary Consolidation (tp) (years):** 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs' (Gvs/σ'v)h-1	Cas	Time after end of Primary Consolidation (years)					
		From	To		Vertical Eff. Stress (kPa)		σ'v w/o surcharge	σ'v			Settlement due to Secondary Consolidation (mm)					
					σ'vs with surcharge	σ'vs										
1	CL-ML	3.5	3.6	0.1	205.52	147.22	0.39600598	0.0036	0.0	0.0	0.1	0.1	0.1	0.1	0.1	0.1
2	CL-ML	3.6	3.8	0.2	207.85	149.95	0.38612871	0.0036	0.0	0.1	0.1	0.1	0.2	0.2	0.3	0.3
3	CL-ML	3.8	4.05	0.25	211.27	154.05	0.37143784	0.0036	0.0	0.1	0.2	0.2	0.2	0.4	0.4	0.4
4	CL-ML	4.05	4.35	0.3	213.39	157.08	0.35847976	0.0036	0.0	0.1	0.2	0.3	0.3	0.4	0.4	0.4
5	CL-ML	4.35	4.75	0.4	215.11	159.98	0.34460558	0.0036	0.0	0.1	0.3	0.4	0.4	0.6	0.6	0.6
6	CL-ML	4.75	5.15	0.4	217.3	163.25	0.33108729	0.0036	0.0	0.1	0.3	0.4	0.4	0.6	0.6	0.6
7	CL-ML	5.15	5.45	0.3	219.42	166.09	0.32109097	0.0036	0.0	0.1	0.2	0.3	0.3	0.5	0.5	0.5
8	CL-ML	5.45	5.7	0.25	221.09	168.3	0.31366607	0.0036	0.0	0.1	0.2	0.3	0.3	0.4	0.4	0.4
9	CL-ML	5.7	5.9	0.2	222.41	170.1	0.30752499	0.0036	0.0	0.1	0.2	0.2	0.2	0.3	0.3	0.3
10	CL-ML	5.9	6	0.1	223.57	171.3	0.30513719	0.0036	0.0	0.0	0.1	0.1	0.1	0.2	0.2	0.2
11	Sand	6	7.5	1.5	227.72	177.55	0.28256829	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
12	CL	7.5	7.6	0.1	231.62	183.71	0.26079146	0.0044	0.0	0.1	0.1	0.2	0.2	0.3	0.3	0.3
13	CL	7.6	7.8	0.2	232.34	184.78	0.25738716	0.0044	0.0	0.1	0.2	0.3	0.3	0.5	0.5	0.5
14	CL	7.8	8.2	0.4	233.79	186.92	0.25074898	0.0044	0.1	0.3	0.5	0.7	1.1	1.1	1.1	1.1
15	CL	8.2	8.4	0.2	235.25	189.05	0.24437979	0.0044	0.0	0.2	0.3	0.4	0.5	0.5	0.5	0.5
16	CL	8.4	8.5	0.1	235.98	190.11	0.24128136	0.0044	0.0	0.1	0.1	0.2	0.2	0.3	0.3	0.3
									0.3	1.6	3.0	4.3	6.5	6.5	6.5	6.5



**Job Number:** 15-64-15  
**Job Description:** Hwy69 - Four-Laning - Estaire  
**Case:** SBL North Approach- 3 - [Rockfill + Surcharge] to Rockfill - Case A to Case B - O/C

**Duration of Primary Consolidation (tp) (years):** 0.5      years       $\longrightarrow$       6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs' (σ <sub>vs</sub> /σ' <sub>vf</sub> )-1	C <sub>αs</sub>	Time after end of Primary Consolidation (years)					
		From	To		Vertical Eff. Stress (kPa)		σ' <sub>vs</sub> with surcharge	σ' <sub>vf</sub> w/o surcharge			1	3	6	10	20	
1	CL-ML	3.5	3.6	0.1	204.88		146.46		0.39888024	0.0036	0.0	0.0	0.1	0.1	0.1	0.1
2	CL-ML	3.6	3.8	0.2	207.2		149.2		0.38873995	0.0036	0.0	0.1	0.1	0.2	0.2	0.3
3	CL-ML	3.8	4.05	0.25	210.62		153.29		0.373997	0.0036	0.0	0.1	0.2	0.2	0.3	0.3
4	CL-ML	4.05	4.35	0.3	212.77		156.32		0.36111822	0.0036	0.0	0.1	0.2	0.3	0.4	0.4
5	CL-ML	4.35	4.75	0.4	214.56		159.22		0.3475694	0.0036	0.0	0.1	0.3	0.4	0.6	0.6
6	CL-ML	4.75	5.15	0.4	216.75		162.49		0.33392824	0.0036	0.0	0.1	0.3	0.4	0.6	0.6
7	CL-ML	5.15	5.45	0.3	218.8		165.33		0.32341378	0.0036	0.0	0.1	0.2	0.3	0.5	0.5
8	CL-ML	5.45	5.7	0.25	220.4		167.54		0.31550674	0.0036	0.0	0.1	0.2	0.3	0.4	0.4
9	CL-ML	5.7	5.9	0.2	221.69		169.34		0.30914137	0.0036	0.0	0.1	0.2	0.2	0.3	0.3
10	CL-ML	5.9	6	0.1	222.76		170.53		0.30628042	0.0036	0.0	0.0	0.1	0.1	0.2	0.2
11	Sand	6	7.5	1.5	226.92		176.8		0.28348416	0	0.0	0.0	0.0	0.0	0.0	0.0
12	CL	7.5	7.6	0.1	230.89		182.96		0.26196983	0.0044	0.0	0.1	0.1	0.2	0.3	0.3
13	CL	7.6	7.8	0.2	231.62		184.04		0.25853075	0.0044	0.0	0.1	0.2	0.3	0.5	0.5
14	CL	7.8	8.2	0.4	233.06		186.18		0.25179933	0.0044	0.1	0.3	0.5	0.7	1.0	1.0
15	CL	8.2	8.4	0.2	234.52		188.31		0.24539323	0.0044	0.0	0.1	0.3	0.4	0.5	0.5
16	CL	8.4	8.5	0.1	235.26		189.37		0.24232983	0.0044	0.0	0.1	0.1	0.2	0.3	0.3
											0.3	1.6	3.0	4.3	6.5	6.5

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: NBL North Approach- 2 - [Sandfill + Surcharge] to Sandfill - Case C to Case D - N/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs ( $\sigma_v/\sigma_{vf}$ )-1	Case	Time at end of Primary Consolidation (years)				
		From	To		Vertical Eff. Stress (kPa) With surcharge	$\sigma_v$	Vertical Eff. Stress (kPa) w/o surcharge	$\sigma_{vf}$			Settlement due to Secondary Consolidation (mm)				
1	CL-ML	4	5.5	1.5	224.92	168.95	0.33128144	0.0036	0.1	0.1	0.6	1.1	1.6	2.4	2.4
2	CL-ML	5.5	7	1.5	230.67	180.64	0.2769597	0.0036	0.1	0.1	0.8	1.4	2.0	2.9	2.9
3	Sand	7	10	3	251.25	198.47	0.2669344	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	CL-CL	10	11	1	256.33	213.26	0.20196005	0.0044	0.3	0.3	1.0	1.6	2.2	3.2	3.2
5	CL-CL	11	13	2	257.29	223.13	0.15309461	0.0044	1.9	1.9	4.1	5.8	7.3	9.5	9.5
6	CL-CL	13	17	4	275.56	243.56	0.13138446	0.0044	5.3	5.3	10.5	14.3	17.5	22.1	22.1
7	CL-CL	17	19	2	293.71	264.18	0.11177985	0.0044	3.5	3.5	6.4	8.5	10.1	12.6	12.6
8	CL-CL	19	20	1	302.45	274.62	0.10134003	0.0044	2.0	2.0	3.5	4.6	5.5	6.7	6.7
9	CL-CH	20	21	1	307.94	281.15	0.09528721	0.005	2.5	2.5	4.3	5.5	6.5	7.9	7.9
10	CL-CH	21	23	2	315.69	290.29	0.08749871	0.005	5.5	5.5	9.2	11.8	13.8	16.6	16.6
11	CL-CH	23	26	3	329.12	305.81	0.0762238	0.005	9.5	9.5	15.3	19.3	22.4	26.7	26.7
12	CL-CH	26	28	2	343.02	321.63	0.06650499	0.005	7.1	7.1	11.2	13.9	16.0	18.9	18.9
13	CL-CH	28	29	1	351.67	331.28	0.05154914	0.005	3.8	3.8	5.9	7.2	8.3	9.8	9.8
14	CL-ML	29	30	1	358.01	338.53	0.05754291	0.004	3.2	3.2	4.9	6.0	6.8	8.0	8.0
15	CL-ML	30	33	3	371.75	354.69	0.04809834	0.004	10.8	10.8	16.0	19.4	22.0	25.6	25.6
16	CL-ML	33	34	1	399.72	371.4	0.04932687	0.004	3.6	3.6	5.3	6.4	7.3	8.5	8.5
									59.1	59.1	98.8	126.9	149.2	181.5	181.5

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: NBL North Approach- 2 - [Sandfill + Surcharge] to Sandfill - Case C to Case D - O/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs ( $\sigma'_{vs}/\sigma'_{vf}$ )-1	C $\alpha$	Time after end of Primary Consolidation (years)					
		From	To		Vertical Eff. Stress (kPa)		$\sigma'_{vf}$ w/o surcharge	Settlement due to Secondary Consolidation (mm)								
					$\sigma'_{vs}$ with surcharge	$\sigma'_{vf}$		1			3	6	10	20		
1	CL-ML	4	5.5	1.5	224.01	167.87	0.33442545	0.0036	0.1	0.5	1.1	1.6	2.4			
2	CL-ML	5.5	7	1.5	230.04	179.88	0.27885257	0.0036	0.1	0.7	1.4	1.9	2.9			
3	Sand	7	10	3	249.96	197.45	0.26594074	0	0.0	0.0	0.0	0.0	0.0			
4	CL-Cl	10	11	1	255.5	212.3	0.20348563	0.0044	0.3	1.0	1.6	2.2	3.2			
5	CL-Cl	11	13	2	256.8	222.52	0.15405357	0.0044	1.8	4.0	5.8	7.2	9.4			
6	CL-Cl	13	17	4	274.86	243.04	0.13092495	0.0044	5.3	10.5	14.4	17.5	22.2			
7	CL-Cl	17	19	2	293.29	263.77	0.11191568	0.0044	3.5	6.4	8.5	10.1	12.6			
8	CL-Cl	19	20	1	302.09	274.25	0.10151322	0.0044	2.0	3.5	4.6	5.5	6.7			
9	Cl-CH	20	21	1	307.61	280.81	0.0954382	0.005	2.5	4.3	5.5	6.5	7.9			
10	Cl-CH	21	23	2	315.41	289.99	0.0876582	0.005	5.5	9.2	11.7	13.7	16.6			
11	Cl-CH	23	26	3	328.89	305.58	0.07628117	0.005	9.5	15.3	19.3	22.4	26.7			
12	Cl-CH	26	28	2	342.8	321.46	0.06638462	0.005	7.1	11.2	13.9	16.0	19.0			
13	Cl-CH	28	29	1	351.49	331.14	0.06145437	0.005	3.8	5.9	7.2	8.3	9.8			
14	CL-ML	29	30	1	357.86	338.41	0.05747466	0.004	3.2	4.9	6.0	6.8	8.0			
15	CL-ML	30	33	3	371.91	354.61	0.04878599	0.004	10.7	15.9	19.3	21.9	25.5			
16	CL-ML	33	34	1	389.37	371.05	0.0493734	0.004	3.6	5.3	6.4	7.3	8.5			
									59.0	98.6	126.8	149.0	181.3			

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: NBL North Approach- 2 - Sandfill to Geofoam - Case D to Case E - N/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=6m)		(H=6m)		Rs (σ'vs/σ'v)-1	Case	Time after end of Primary Consolidation (years)					
		From	To		σ'vs with surcharge	Vertical Eff. Stress (kPa)	σ'vf w/o surcharge	1			3	6	10	20		
Settlement due to Secondary Consolidation (mm)																
1	CL-ML	4	5.5	1.5	168.95	115.67	0.46062073	0.0036	0.0	0.0	0.3	0.6	1.0	1.6		
2	CL-ML	5.5	7	1.5	180.64	128.63	0.40433802	0.0036	0.0	0.0	0.4	0.8	1.2	1.9		
3	Sand	7	10	3	198.47	147.9	0.34192022	0	0.0	0.0	0.0	0.0	0.0	0.0		
4	CL-Cl	10	11	1	213.26	164.61	0.29554705	0.0044	0.1	0.1	0.5	1.0	1.5	2.2		
5	CL-Cl	11	13	2	223.13	176.5	0.26419263	0.0044	0.3	0.3	1.3	2.4	3.4	5.0		
6	CL-Cl	13	17	4	243.56	200.37	0.21555123	0.0044	0.9	0.9	3.6	6.1	8.4	12.1		
7	CL-Cl	17	19	2	264.18	224.35	0.1775351	0.0044	1.1	1.1	2.9	4.5	5.8	7.9		
8	CL-Cl	19	20	1	274.62	236.41	0.16162599	0.0044	0.8	0.8	1.8	2.7	3.4	4.5		
9	Cl-CH	20	21	1	281.15	243.98	0.15234855	0.005	1.1	1.1	2.3	3.3	4.2	5.4		
10	Cl-CH	21	23	2	290.29	254.63	0.14004634	0.005	2.6	2.6	5.4	7.5	9.3	11.9		
11	Cl-CH	23	26	3	305.81	272.48	0.1223209	0.005	5.2	5.2	9.8	13.2	16.0	20.0		
12	Cl-CH	26	28	2	321.63	290.47	0.10727442	0.005	4.2	4.2	7.6	10.0	11.9	14.7		
13	Cl-CH	28	29	1	331.28	301.32	0.09942918	0.005	2.3	2.3	4.1	5.3	6.3	7.7		
14	CL-ML	29	30	1	338.53	309.32	0.09443295	0.004	2.0	2.0	3.4	4.4	5.2	6.4		
15	CL-ML	30	33	3	354.69	326.89	0.0850439	0.004	6.8	6.8	11.3	14.4	16.8	20.2		
16	CL-ML	33	34	1	371.4	344.55	0.07792773	0.004	2.5	2.5	4.0	5.1	5.9	7.1		
									30.0		59.0	81.5	100.3	128.5		

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: NBL North Approach- 2 - Sandfill to Geofoam - Case D to Case E - O/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=6m)		Vertical Eff. Stress (kPa)	(H=6m)	Rs' (σ'vs/σ'v)-1	Coe	Time after end of Primary Consolidation (years)					
		From	To		σ'vs with surcharge	σ'v w/o surcharge					1	3	6	10	20	
Settlement due to Secondary Consolidation (mm)																
1	CL-ML	4	5.5	1.5	167.87	115.56	0.45266528	0.0036	0.0	0.3	0.7	1.0	1.6			
2	CL-ML	5.5	7	1.5	179.88	128.55	0.39929988	0.0036	0.0	0.4	0.8	1.2	1.9			
3	Sand	7	10	3	197.45	147.8	0.33592693	0	0.0	0.0	0.0	0.0	0.0			
4	CL-Cl	10	11	1	212.3	164.51	0.29049906	0.0044	0.1	0.6	1.0	1.5	2.2			
5	CL-Cl	11	13	2	222.52	176.44	0.26116527	0.0044	0.3	1.4	2.4	3.4	5.0			
6	CL-Cl	13	17	4	243.04	200.31	0.21331935	0.0044	1.0	3.6	6.2	8.5	12.3			
7	CL-Cl	17	19	2	263.77	224.31	0.17591726	0.0044	1.2	3.0	4.6	5.9	8.0			
8	CL-Cl	19	20	1	274.25	236.37	0.16025722	0.0044	0.8	1.9	2.7	3.4	4.5			
9	Cl-CH	20	21	1	280.81	243.94	0.15114372	0.005	1.1	2.4	3.4	4.2	5.5			
10	Cl-CH	21	23	2	289.99	254.6	0.13900236	0.005	2.7	5.5	7.6	9.3	11.9			
11	Cl-CH	23	26	3	305.58	272.46	0.12155913	0.005	5.2	9.9	13.3	16.1	20.1			
12	Cl-CH	26	28	2	321.46	290.45	0.10676536	0.005	4.3	7.7	10.1	12.0	14.7			
13	Cl-CH	28	29	1	331.14	301.3	0.0990375	0.005	2.4	4.1	5.4	6.3	7.7			
14	CL-ML	29	30	1	338.41	309.31	0.09408037	0.004	2.0	3.5	4.5	5.2	6.4			
15	CL-ML	30	33	3	354.61	326.88	0.08483235	0.004	6.8	11.3	14.4	16.8	20.3			
16	CL-ML	33	34	1	371.05	344.51	0.07703695	0.004	2.5	4.1	5.1	5.9	7.1			
									30.3	59.5	82.1	101.0	129.4			

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: NBL North Approach- 2 - Geofoam - Case E to Case E - N/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	Vertical Eff. Stress (kPa)		Rs' ( $\sigma'_{vs}/\sigma'_{vf}$ )-1	Cac	Time after end of Primary Consolidation (years)							
		From	To		with surcharge				$\sigma'_{vs}$	$\sigma'_{vf}$ w/o surcharge	1	3	6	10	20	
Settlement due to Secondary Consolidation (mm)																
1	CL-ML	4	5.5	1.5	115.67	115.67	0	0.0036	2.6	4.6	6.0	7.1	8.7			
2	CL-ML	5.5	7	1.5	128.63	128.63	0	0.0036	2.6	4.6	6.0	7.1	8.7			
3	Sand	7	10	3	147.9	147.9	0	0	0.0	0.0	0.0	0.0	0.0			
4	CL-Cl	10	11	1	164.61	164.61	0	0.0044	2.1	3.7	4.9	5.8	7.1			
5	CL-Cl	11	13	2	176.5	176.5	0	0.0044	4.2	7.4	9.8	11.6	14.2			
6	CL-Cl	13	17	4	200.37	200.37	0	0.0044	8.4	14.9	19.6	23.3	28.4			
7	CL-Cl	17	19	2	224.35	224.35	0	0.0044	4.2	7.4	9.8	11.6	14.2			
8	CL-Cl	19	20	1	236.41	236.41	0	0.0044	2.1	3.7	4.9	5.8	7.1			
9	Cl-CH	20	21	1	243.98	243.98	0	0.005	2.4	4.2	5.6	6.6	8.1			
10	Cl-CH	21	23	2	254.63	254.63	0	0.005	4.8	8.5	11.1	13.2	16.1			
11	Cl-CH	23	26	3	272.48	272.48	0	0.005	7.2	12.7	16.7	19.8	24.2			
12	Cl-CH	26	28	2	290.47	290.47	0	0.005	4.8	8.5	11.1	13.2	16.1			
13	Cl-CH	28	29	1	301.32	301.32	0	0.005	2.4	4.2	5.6	6.6	8.1			
14	CL-ML	29	30	1	309.32	309.32	0	0.004	1.9	3.4	4.5	5.3	6.5			
15	CL-ML	30	33	3	326.89	326.89	0	0.004	5.7	10.1	13.4	15.9	19.4			
16	CL-ML	33	34	1	344.55	344.55	0	0.004	1.9	3.4	4.5	5.3	6.5			
								57.2	101.2	133.5	158.4	193.2				

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: NBL North Approach- 2 - Geofoam - Case E to Case E - O/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	Vertical Eff. Stress (kPa)		Rs ( $\sigma_v/\sigma'_{vf}-1$ )	C $\sigma_c$	Time after end of Primary Consolidation (years)					
		From	To		$\sigma'_v$ s with surcharge	$\sigma'_v$ f w/o surcharge			1	3	6	10	20	
1	CL-ML	4	5.5	1.5	115.56	115.56	0	0.0036	2.6	4.6	6.0	7.1	8.7	
2	CL-ML	5.5	7	1.5	128.55	128.55	0	0.0036	2.6	4.6	6.0	7.1	8.7	
3	Sand	7	10	3	147.8	147.8	0	0	0.0	0.0	0.0	0.0	0.0	
4	CL-Cl	10	11	1	164.51	164.51	0	0.0044	2.1	3.7	4.9	5.8	7.1	
5	CL-Cl	11	13	2	176.44	176.44	0	0.0044	4.2	7.4	9.8	11.6	14.2	
6	CL-Cl	13	17	4	200.31	200.31	0	0.0044	8.4	14.9	19.6	23.3	28.4	
7	CL-Cl	17	19	2	224.31	224.31	0	0.0044	4.2	7.4	9.8	11.6	14.2	
8	CL-Cl	19	20	1	236.37	236.37	0	0.0044	2.1	3.7	4.9	5.8	7.1	
9	Cl-CH	20	21	1	244.07	243.94	0.00053292	0.005	0.0	0.0	0.0	0.0	0.0	
10	Cl-CH	21	23	2	256.76	254.6	0.0084839	0.005	0.0	0.0	0.0	0.0	0.0	
11	Cl-CH	23	26	3	277.9	272.46	0.01996623	0.005	0.0	0.0	0.0	0.0	0.0	
12	Cl-CH	26	28	2	299.06	290.45	0.02964366	0.005	0.0	0.0	0.0	0.0	0.0	
13	Cl-CH	28	29	1	311.74	301.3	0.03464985	0.005	0.0	0.0	0.0	0.0	0.0	
14	CL-ML	29	30	1	309.31	309.31	0	0.004	1.9	3.4	4.5	5.3	6.5	
15	CL-ML	30	33	3	326.88	326.88	0	0.004	5.7	10.1	13.4	15.9	19.4	
16	CL-ML	33	34	1	345.03	344.51	0.00150939	0.004	0.0	0.0	0.0	0.0	0.0	
greater of pre-consolidation pressure or $\sigma'_v$ s									33.8	59.8	78.9	93.6	114.2	

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: NBL North Approach - 3 - [Rockfill + Surcharge] to Rockfill - Case A to Case B - N/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs' (σvs/σ'v)-1	C <sub>as</sub>	Time at end of Primary Consolidation (years)					
		From	To		Vertical Eff. Stress (kPa)		(H=6m)				Settlement due to Secondary Consolidation (mm)	1	3	6	10	20
					σ <sub>vs</sub> with surcharge	σ <sub>vf</sub> w/o surcharge	σ <sub>vs</sub>	σ <sub>vf</sub>								
1	CL-ML	5.5	5.7	0.2	219.5	167.7	169.93	0.3088491	0.0036	0.0	0.1	0.2	0.2	0.3		
2	CL-ML	5.7	6.1	0.4	220.47	169.93	174.66	0.29741658	0.0036	0.0	0.2	0.3	0.5	0.7		
3	CL-ML	6.1	6.9	0.8	223.39	174.66	179.33	0.2789992	0.0036	0.1	0.4	0.7	1.0	1.5		
4	CL-ML	6.9	7.3	0.4	226.71	179.33	181.64	0.26420565	0.0036	0.0	0.2	0.4	0.6	0.8		
5	CL-ML	7.3	7.5	0.2	228.6	181.64	182.77	0.25853336	0.0036	0.0	0.1	0.2	0.3	0.4		
6	CL-CI	7.5	7.6	0.1	229.41	182.77	183.84	0.25518411	0.0044	0.0	0.1	0.1	0.2	0.3		
7	CL-CI	7.6	7.8	0.2	230.08	183.84	185.97	0.25152306	0.0044	0.0	0.1	0.3	0.4	0.5		
8	CL-CI	7.8	8.2	0.4	231.38	185.97	190.2	0.24417917	0.0044	0.1	0.3	0.5	0.7	1.1		
9	CL-CI	8.2	9	0.8	234.19	190.2	197.17	0.23128286	0.0044	0.2	0.7	1.1	1.6	2.3		
10	CL-CI	9	10.2	1.2	239.07	197.17	206.47	0.21250697	0.0044	0.3	1.1	1.9	2.6	3.7		
11	CL-CI	10.2	11.7	1.5	245.4	206.47	215.71	0.18555039	0.0044	0.6	1.8	2.9	3.9	5.4		
12	CL-CI	11.7	12.9	1.2	251.32	215.71	222.55	0.16508275	0.0044	0.9	2.1	3.1	3.9	5.2		
13	CL-CI	12.9	13.7	0.8	256.53	222.55	226.66	0.15268479	0.0044	0.7	1.6	2.3	2.9	3.8		
14	CL-CI	13.7	14.1	0.4	260.62	226.66	228.72	0.14982794	0.0044	0.4	0.9	1.2	1.5	1.9		
15	CL-CI	14.1	14.3	0.2	262.61	228.72	229.92	0.14817244	0.0044	0.2	0.4	0.6	0.8	1.0		
16	CL-CI	14.3	14.4	0.1	263.94	229.92		0.14796451	0.0044	0.1	0.2	0.3	0.4	0.5		
											3.7	10.4	16.2	21.4	29.5	



**Job Number:** 15-64-15  
**Job Description:** Hwy69 - Four-Laning - Estaire  
**Case:** NBL North Approach - 3 - [Rockfill + Surcharge] to Rockfill - Case A to Case B - O/C

**Duration of Primary Consolidation (tp) (years):** 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs (σ <sub>v</sub> /σ <sub>v'</sub> )-1	C <sub>α</sub>	Time at end of Primary Consolidation (years)					
		From	To		Vertical Eff. Stress (kPa)						Settlement due to Secondary Consolidation (mm)					
					σ <sub>v</sub> with surcharge	σ <sub>v'</sub> w/o surcharge	σ <sub>v'</sub>									
1	CL-ML	5.5	5.7	0.2	219.44	167.51	0.31001134	0.0036	0.0	0.1	0.2	0.2	0.2	0.3		
2	CL-ML	5.7	6.1	0.4	220.45	169.81	0.29821565	0.0036	0.0	0.2	0.3	0.3	0.5	0.7		
3	CL-ML	6.1	6.9	0.8	223.37	174.54	0.27976395	0.0036	0.1	0.4	0.7	1.0	1.0	1.5		
4	CL-ML	6.9	7.3	0.4	226.69	179.22	0.26486999	0.0036	0.0	0.2	0.4	0.6	0.6	0.8		
5	CL-ML	7.3	7.5	0.2	228.56	181.53	0.25907563	0.0036	0.0	0.1	0.2	0.3	0.3	0.4		
6	CL-Cl	7.5	7.6	0.1	229.37	182.66	0.25572101	0.0044	0.0	0.1	0.1	0.2	0.2	0.3		
7	CL-Cl	7.6	7.8	0.2	230.05	183.73	0.25210907	0.0044	0.0	0.1	0.3	0.4	0.4	0.5		
8	CL-Cl	7.8	8.2	0.4	231.35	185.86	0.24475412	0.0044	0.1	0.3	0.5	0.7	1.1	1.1		
9	CL-Cl	8.2	9	0.8	234.11	190.09	0.23157452	0.0044	0.2	0.7	1.1	1.6	2.3	2.3		
10	CL-Cl	9	10.2	1.2	238.93	197.07	0.21241183	0.0044	0.3	1.1	1.9	2.6	3.7	3.7		
11	CL-Cl	10.2	11.7	1.5	245.22	206.37	0.18825411	0.0044	0.6	1.9	2.9	3.9	5.4	5.4		
12	CL-Cl	11.7	12.9	1.2	251.3	215.63	0.16542225	0.0044	0.9	2.1	3.1	3.9	5.2	5.2		
13	CL-Cl	12.9	13.7	0.8	256.65	222.47	0.15363869	0.0044	0.7	1.6	2.3	2.9	3.8	3.8		
14	CL-Cl	13.7	14.1	0.4	260.64	226.58	0.15032218	0.0044	0.4	0.8	1.2	1.5	1.9	1.9		
15	CL-Cl	14.1	14.3	0.2	262.57	228.64	0.14839923	0.0044	0.2	0.4	0.6	0.8	1.0	1.0		
16	CL-Cl	14.3	14.4	0.1	263.84	229.77	0.14827871	0.0044	0.1	0.2	0.3	0.4	0.5	0.5		
									3.7	10.3	16.2	21.3	29.5	29.5		

**Job Number:** 15-64-15  
**Job Description:** Hwy69 - Four-Laning - Estaire  
**Case:** NBL North Approach- 3 - [Sandfill + Surcharge] to Sandfill - Case C to Case D - N/C

**Duration of Primary Consolidation (tp) (years):** 0.5 years → 6 months

										Time after end of Primary Consolidation (years)														
										1	3	6	10	20										
Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs ( $\sigma'_{vs}/\sigma'_{v1}$ -1)	Case	Settlement due to Secondary Consolidation (mm)													
		From	To		$\sigma'_{vs}$ with surcharge	Vertical Eff. Stress (kPa)	$\sigma'_{v1}$ w/o surcharge																	
1	CL-ML	5.5	5.7	0.2	235.95	184.04	0.28205825	0.0036	0.0036	0.0	0.1	0.2	0.3	0.4	0.8									
2	CL-ML	5.7	6.1	0.4	236.81	186.24	0.27153136	0.0036	0.0036	0.0	0.2	0.4	0.5	0.8	1.7									
3	CL-ML	6.1	6.9	0.8	239.68	190.99	0.25493481	0.0036	0.0036	0.1	0.5	0.8	1.2	1.7										
4	CL-ML	6.9	7.3	0.4	243.04	195.67	0.24209128	0.0036	0.0036	0.1	0.2	0.4	0.6	0.9										
5	CL-ML	7.3	7.5	0.2	244.98	197.98	0.23739772	0.0036	0.0036	0.0	0.1	0.2	0.3	0.5										
6	CL-Cl	7.5	7.6	0.1	245.81	199.11	0.23454372	0.0044	0.0044	0.0	0.1	0.1	0.2	0.3										
7	CL-Cl	7.6	7.8	0.2	246.46	200.18	0.23119193	0.0044	0.0044	0.0	0.2	0.3	0.4	0.6										
8	CL-Cl	7.8	8.2	0.4	247.73	202.31	0.22450694	0.0044	0.0044	0.1	0.3	0.6	0.8	1.2										
9	CL-Cl	8.2	9	0.8	250.52	206.53	0.21298569	0.0044	0.0044	0.2	0.7	1.2	1.7	2.5										
10	CL-Cl	9	10.2	1.2	255.36	213.45	0.19634575	0.0044	0.0044	0.4	1.3	2.1	2.8	4.0										
11	CL-Cl	10.2	11.7	1.5	261.51	222.66	0.17448127	0.0044	0.0044	0.9	2.3	3.5	4.5	6.1										
12	CL-Cl	11.7	12.9	1.2	267.05	231.76	0.15226959	0.0044	0.0044	1.1	2.5	3.5	4.4	5.7										
13	CL-Cl	12.9	13.7	0.8	272.12	238.47	0.1411079	0.0044	0.0044	0.9	1.9	2.6	3.2	4.1										
14	CL-Cl	13.7	14.1	0.4	276.37	242.49	0.1397171	0.0044	0.0044	0.5	1.0	1.3	1.6	2.1										
15	CL-Cl	14.1	14.3	0.2	278.41	244.5	0.13869121	0.0044	0.0044	0.2	0.5	0.7	0.8	1.1										
16	CL-Cl	14.3	14.4	0.1	279.81	245.71	0.13878149	0.0044	0.0044	0.1	0.2	0.3	0.4	0.5										
										4.7	12.1	18.4	23.8	32.3										

**Job Number:** 15-64-15  
**Job Description:** Hwy69 - Four-Laning - Estaire  
**Case:** NBL North Approach- 3 - [Sandfill + Surcharge] to Sandfill - Case C to Case D - O/C

**Duration of Primary Consolidation (tp) (years):** 0.5 years  $\longrightarrow$  6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=9m)		(H=6m)		Rs' (σ <sub>v</sub> /σ <sub>v'</sub> ) <sup>-1</sup>	C <sub>as</sub>	Time after end of Primary Consolidation (years)						
		From	To		Vertical Eff. Stress (kPa)		σ <sub>v</sub> /σ <sub>v'</sub> w/o surcharge	σ <sub>v</sub> /σ <sub>v'</sub> with surcharge			1	2	3	6	10	20	
Settlement due to Secondary Consolidation (mm)																	
1	CL-ML	5.5	5.7	0.2	236.85	183.79	0.28325807	0.0036	0.0	0.1	0.2	0.2	0.3	0.4			
2	CL-ML	5.7	6.1	0.4	236.73	186.06	0.27233151	0.0036	0.0	0.2	0.4	0.4	0.5	0.8			
3	CL-ML	6.1	6.9	0.8	239.61	190.81	0.25575179	0.0036	0.1	0.5	0.8	0.8	1.1	1.7			
4	CL-ML	6.9	7.3	0.4	242.96	195.49	0.24282572	0.0036	0.1	0.2	0.4	0.4	0.6	0.9			
5	CL-ML	7.3	7.5	0.2	244.89	197.8	0.23806876	0.0036	0.0	0.1	0.2	0.2	0.3	0.5			
6	CL-CI	7.5	7.6	0.1	245.72	198.93	0.23520836	0.0044	0.0	0.1	0.1	0.1	0.2	0.3			
7	CL-CI	7.6	7.8	0.2	246.38	200	0.2319	0.0044	0.0	0.2	0.3	0.3	0.4	0.6			
8	CL-CI	7.8	8.2	0.4	247.66	202.13	0.22525108	0.0044	0.1	0.3	0.6	0.6	0.8	1.2			
9	CL-CI	8.2	9	0.8	250.38	206.35	0.21337533	0.0044	0.2	0.7	1.2	1.2	1.7	2.5			
10	CL-CI	9	10.2	1.2	255.16	213.29	0.1963055	0.0044	0.4	1.3	2.1	2.1	2.8	4.0			
11	CL-CI	10.2	11.7	1.5	261.26	222.5	0.17420225	0.0044	0.9	2.3	3.5	3.5	4.5	6.1			
12	CL-CI	11.7	12.9	1.2	266.98	231.61	0.15271361	0.0044	1.1	2.5	3.5	3.5	4.4	5.7			
13	CL-CI	12.9	13.7	0.8	272.21	238.33	0.14215883	0.0044	0.9	1.9	2.6	2.6	3.2	4.1			
14	CL-CI	13.7	14.1	0.4	276.33	242.35	0.14021044	0.0044	0.5	1.0	1.3	1.3	1.6	2.1			
15	CL-CI	14.1	14.3	0.2	278.31	244.36	0.13893436	0.0044	0.2	0.5	0.7	0.7	0.8	1.1			
16	CL-CI	14.3	14.4	0.1	279.67	245.49	0.13923174	0.0044	0.1	0.2	0.3	0.3	0.4	0.5			
									4.7	12.0	18.3	23.8	32.3				

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: NBL North Approach- 3 - Sandfill to Geofam - Case D to Case E - N/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=6m)		(H=6m)		Rs (σ <sub>v</sub> /σ <sub>v</sub> ) <sup>-1</sup>	Cas	Time after end of Primary Consolidation (years)							
		From	To		σ <sub>v</sub> with surcharge	Vertical Eff. Stress (kPa)	σ <sub>v</sub> w/o surcharge	1			3	6	10	20	Settlement due to Secondary Consolidation (mm)			
1	CL-ML	5.5	5.7	0.2	184.04	124.17	124.17	0.48216155	0.0036	0.0036	0.0	0.0	0.1	0.1	0.1	0.2		
2	CL-ML	5.7	6.1	0.4	186.24	126.74	126.74	0.46946505	0.0036	0.0036	0.0	0.0	0.1	0.2	0.3	0.4		
3	CL-ML	6.1	6.9	0.8	190.99	131.9	131.9	0.4479909	0.0036	0.0036	0.0	0.0	0.2	0.4	0.6	0.9		
4	CL-ML	6.9	7.3	0.4	195.67	137.04	137.04	0.42783129	0.0036	0.0036	0.0	0.0	0.1	0.2	0.3	0.5		
5	CL-ML	7.3	7.5	0.2	197.98	139.61	139.61	0.41809326	0.0036	0.0036	0.0	0.0	0.0	0.1	0.2	0.2		
6	CL-Cl	7.5	7.6	0.1	199.11	140.86	140.86	0.41353117	0.0044	0.0044	0.0	0.0	0.0	0.1	0.1	0.2		
7	CL-Cl	7.6	7.8	0.2	200.18	142.07	142.07	0.40902372	0.0044	0.0044	0.0	0.0	0.1	0.1	0.2	0.3		
8	CL-Cl	7.8	8.2	0.4	202.31	144.47	144.47	0.40035994	0.0044	0.0044	0.0	0.0	0.1	0.3	0.4	0.6		
9	CL-Cl	8.2	9	0.8	206.53	149.27	149.27	0.38360019	0.0044	0.0044	0.0	0.0	0.3	0.6	0.9	1.3		
10	CL-Cl	9	10.2	1.2	213.45	157.25	157.25	0.35739269	0.0044	0.0044	0.1	0.5	0.9	1.4	2.0	2.2		
11	CL-Cl	10.2	11.7	1.5	222.66	167.99	167.99	0.32543604	0.0044	0.0044	0.1	0.7	1.3	2.0	3.0	3.0		
12	CL-Cl	11.7	12.9	1.2	231.76	178.71	178.71	0.29684964	0.0044	0.0044	0.1	0.7	1.2	1.7	2.6	2.6		
13	CL-Cl	12.9	13.7	0.8	238.47	186.65	186.65	0.27763193	0.0044	0.0044	0.1	0.5	0.9	1.3	1.9	1.9		
14	CL-Cl	13.7	14.1	0.4	242.49	191.42	191.42	0.26679553	0.0044	0.0044	0.1	0.3	0.5	0.7	1.0	1.0		
15	CL-Cl	14.1	14.3	0.2	244.5	193.8	193.8	0.26160991	0.0044	0.0044	0.0	0.1	0.2	0.3	0.5	0.5		
16	CL-Cl	14.3	14.4	0.1	245.71	195.01	195.01	0.25986867	0.0044	0.0044	0.0	0.1	0.1	0.2	0.3	0.3		
											0.5	3.7	7.2	10.5	16.0	16.0		

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: NBL North Approach- 3 - Sandfill to Geofam - Case D to Case E - O/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	(H=6m)		Rs ( $\sigma'_{vs}/\sigma'_{vf}$ )-1	Case	Time after end of Primary Consolidation (years)						
		From	To		Vertical Eff. Stress (kPa)				0.0	0.1	0.2	0.3	6	10	20
					c'vs With surcharge	$\sigma'_{vf}$ w/o surcharge									
1	CL-ML	5.5	5.7	0.2	183.79	124.11	0.48086375	0.0036	0.0	0.0	0.1	0.1	0.2	0.2	0.2
2	CL-ML	5.7	6.1	0.4	186.06	126.7	0.46850829	0.0036	0.0	0.1	0.1	0.2	0.3	0.4	0.4
3	CL-ML	6.1	6.9	0.8	190.81	131.86	0.44708507	0.0036	0.0	0.2	0.2	0.4	0.6	0.9	0.9
4	CL-ML	6.9	7.3	0.4	195.49	137	0.42693431	0.0036	0.0	0.1	0.1	0.2	0.3	0.5	0.5
5	CL-ML	7.3	7.5	0.2	197.8	139.57	0.41721	0.0036	0.0	0.0	0.1	0.1	0.2	0.2	0.2
6	CL-Cl	7.5	7.6	0.1	198.93	140.82	0.41265445	0.0044	0.0	0.0	0.1	0.1	0.1	0.2	0.2
7	CL-Cl	7.6	7.8	0.2	200	142.03	0.40815321	0.0044	0.0	0.1	0.1	0.1	0.2	0.3	0.3
8	CL-Cl	7.8	8.2	0.4	202.13	144.43	0.39950149	0.0044	0.0	0.1	0.1	0.3	0.4	0.6	0.6
9	CL-Cl	8.2	9	0.8	206.35	149.24	0.38267221	0.0044	0.0	0.3	0.3	0.6	0.9	1.3	1.3
10	CL-Cl	9	10.2	1.2	213.29	157.22	0.35663402	0.0044	0.1	0.5	0.5	1.0	1.4	2.2	2.2
11	CL-Cl	10.2	11.7	1.5	222.5	167.95	0.32479905	0.0044	0.1	0.7	0.7	1.4	2.0	3.0	3.0
12	CL-Cl	11.7	12.9	1.2	231.61	178.68	0.29622789	0.0044	0.1	0.7	0.7	1.2	1.8	2.6	2.6
13	CL-Cl	12.9	13.7	0.8	238.33	186.62	0.27708713	0.0044	0.1	0.5	0.5	0.9	1.3	1.9	1.9
14	CL-Cl	13.7	14.1	0.4	242.35	191.39	0.26626261	0.0044	0.1	0.3	0.3	0.5	0.7	1.0	1.0
15	CL-Cl	14.1	14.3	0.2	244.36	193.77	0.26108273	0.0044	0.0	0.1	0.1	0.2	0.3	0.5	0.5
16	CL-Cl	14.3	14.4	0.1	245.49	194.97	0.25911679	0.0044	0.0	0.1	0.1	0.1	0.2	0.3	0.3
									0.5	3.7	7.2	10.5	16.1		

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: NBL North Approach-3 - Geofram - Case E to Case E - N/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	Vertical Eff. Stress (kPa)		Rs <sup>1</sup> ( $\sigma_v/\sigma_{vf}$ )-1	Cat	Time after end of Primary Consolidation (Years)				
		From	To		$\sigma_v$ with surcharge	$\sigma_{vf}$ w/o surcharge			1	3	6	10	20
Settlement due to Secondary Consolidation (mm)													
1	CL-ML	5.5	5.7	0.2	124.17	124.17	0	0.0036	0.3	0.6	0.8	1.0	1.2
2	CL-ML	5.7	6.1	0.4	126.74	126.74	0	0.0036	0.7	1.2	1.6	1.9	2.3
3	CL-ML	6.1	6.9	0.8	131.9	131.9	0	0.0036	1.4	2.4	3.2	3.8	4.6
4	CL-ML	6.9	7.3	0.4	137.04	137.04	0	0.0036	0.7	1.2	1.6	1.9	2.3
5	CL-ML	7.3	7.5	0.2	139.61	139.61	0	0.0036	0.3	0.6	0.8	1.0	1.2
6	CL-CI	7.5	7.6	0.1	140.86	140.86	0	0.0044	0.2	0.4	0.5	0.6	0.7
7	CL-CI	7.6	7.8	0.2	142.07	142.07	0	0.0044	0.4	0.7	1.0	1.2	1.4
8	CL-CI	7.8	8.2	0.4	144.47	144.47	0	0.0044	0.8	1.5	2.0	2.3	2.8
9	CL-CI	8.2	9	0.8	149.27	149.27	0	0.0044	1.7	3.0	3.9	4.7	5.7
10	CL-CI	9	10.2	1.2	157.25	157.25	0	0.0044	2.5	4.5	5.9	7.0	8.5
11	CL-CI	10.2	11.7	1.5	167.99	167.99	0	0.0044	3.1	5.6	7.4	8.7	10.6
12	CL-CI	11.7	12.9	1.2	178.71	178.71	0	0.0044	2.5	4.5	5.9	7.0	8.5
13	CL-CI	12.9	13.7	0.8	186.65	186.65	0	0.0044	1.7	3.0	3.9	4.7	5.7
14	CL-CI	13.7	14.1	0.4	191.42	191.42	0	0.0044	0.8	1.5	2.0	2.3	2.8
15	CL-CI	14.1	14.3	0.2	193.8	193.8	0	0.0044	0.4	0.7	1.0	1.2	1.4
16	CL-CI	14.3	14.4	0.1	195.01	195.01	0	0.0044	0.2	0.4	0.5	0.6	0.7
									17.9	31.7	41.8	49.7	60.6

Job Number: 15-64-15  
 Job Description: Hwy69 - Four-Laning - Estaire  
 Case: NBL North Approach-3 - Geofoam - Case E to Case E - O/C

Duration of Primary Consolidation (tp) (years): 0.5 years → 6 months

Layer	Soil	Depth (m)		Thickness (m)	Vertical Eff. Stress (kPa)		Rs ( $\sigma'_{vs}/\sigma'_{vs0}$ )-1	Gas	Time at end of Primary Consolidation (years)				
		From	To		$\sigma'_{vs}$ with surcharge	$\sigma'_{vs}$ w/o surcharge			1	3	6	10	20
1	CL-ML	5.5	5.7	0.2	136.05	124.11	0.09620498	0.0036	0.0	0.0	0.0	0.0	0.0
2	CL-ML	5.7	6.1	0.4	140.19	126.7	0.10647198	0.0036	0.0	0.0	0.0	0.0	0.0
3	CL-ML	6.1	6.9	0.8	148.46	131.86	0.1258911	0.0036	0.0	0.0	0.0	0.0	0.0
4	CL-ML	6.9	7.3	0.4	156.72	137	0.14394161	0.0036	0.0	0.0	0.0	0.0	0.0
5	CL-ML	7.3	7.5	0.2	160.86	139.57	0.15253994	0.0036	0.0	0.0	0.0	0.0	0.0
6	CL-CI	7.5	7.6	0.1	140.82	140.82	0	0.0044	0.2	0.4	0.5	0.6	0.7
7	CL-CI	7.6	7.8	0.2	142.03	142.03	0	0.0044	0.4	0.7	1.0	1.2	1.4
8	CL-CI	7.8	8.2	0.4	144.43	144.43	0	0.0044	0.8	1.5	2.0	2.3	2.8
9	CL-CI	8.2	9	0.8	149.24	149.24	0	0.0044	1.7	3.0	3.9	4.7	5.7
10	CL-CI	9	10.2	1.2	157.22	157.22	0	0.0044	2.5	4.5	5.9	7.0	8.5
11	CL-CI	10.2	11.7	1.5	167.95	167.95	0	0.0044	3.1	5.6	7.4	8.7	10.6
12	CL-CI	11.7	12.9	1.2	178.68	178.68	0	0.0044	2.5	4.5	5.9	7.0	8.5
13	CL-CI	12.9	13.7	0.8	186.62	186.62	0	0.0044	1.7	3.0	3.9	4.7	5.7
14	CL-CI	13.7	14.1	0.4	191.39	191.39	0	0.0044	0.8	1.5	2.0	2.3	2.8
15	CL-CI	14.1	14.3	0.2	193.77	193.77	0	0.0044	0.4	0.7	1.0	1.2	1.4
16	CL-CI	14.3	14.4	0.1	194.97	194.97	0	0.0044	0.2	0.4	0.5	0.6	0.7
greater of pre-consolidation pressure or $\sigma'_{vs}$									14.5	25.7	33.8	40.1	49.0

## **Appendix I**

### **Non Standard Special Provisions:**

**Granular Blanket**

**Wick Drains**

**Supply and Installation of Instruments (Not included in the Draft Report)**

**Monitoring Program (Not included in the Draft Report)**



**GRANULAR BLANKET Item No. \_\_\_\_**

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**Special Provision**

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**1.0 Scope**

This non-standard special provision specifies the requirements for the surface preparation, supply, placement and compaction of the Granular Blanket in connection with the installation of the prefabricated vertical drains.

**2.0 Materials**

The Granular Blanket shall be Granular'A' material and shall satisfy the physical and gradation requirements as specified in OPSS 1010.

**3.0 Construction**

3.1 The Granular Blanket shall be placed and compacted to the limits and, grades shown on the plans or as directed by the Contract Administrator.

3.2 The Granular Blanket shall be placed subsequent to the required subexcavation. The vertical drains shall not be installed in frozen ground.

3.3 The Granular Blanket shall be end-dumped in areas of land reclamation.

3.4 The Granular Blanket shall be placed and compacted in lift thicknesses not exceeding 150 mm except in land reclamation areas.

3.5 The Granular Blanket shall be compacted to 90%  $\pm$  2% of its standard proctor density.

**4.0 Payment**

**4.1 Measurement of Payment**

Measurement of payment shall be by the tonne. The method of determining the mass of materials for payment shall conform to OPSS 102.

**4.2 Basis of Payment**

Granular Blanket - Item

Payment at the contract price for the above item shall be full compensation for all labour, equipment and material required to do the work.

## **WICK DRAINS - Item No.**

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### **Special Provision**

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#### **1.0 General**

##### **1.1 Scope**

This non-standard special provision specifies the requirements for the supply and installation of wick drains in accordance with the details shown on the plans and with the requirements of these specifications.

##### **1.2 Qualifications**

This work shall be undertaken by a recognized specialist subcontractor, which has completed a minimum of five wick drain installation projects in the last five years, each project with the following characteristics:

- Installation depth: not less than 15m
- Total length of wick drains: not less than 40,000m

The specialist subcontractor's qualification shall be submitted to the Contract Administrator not later than 15 working days in advance of commencing the installation of wick drains.

#### **2.0 Site Conditions**

The Contractor shall refer to the Foundation Investigation Report in the Contract Documents for a description of subsurface conditions at this site. The Record of Borehole sheets are not represented as a complete description of the subsurface conditions, it only present what was found in borings at the indicated locations on the date boreholes were drilled. The subsurface conditions may be variable between the borehole locations. The Contractor should review the existing surface conditions and satisfy himself that his equipment and methodology are appropriate for the installation of wick drains.

#### **3.0 Materials**

The prefabricated drain shall consist of a continuous plastic drainage core wrapped in a non-woven geotextile material. The core configuration should be 'Studded' or 'Grooved' ('Filament' or 'Cuspated' are not acceptable).

The Contractor shall submit samples of the prefabricated drain for evaluation and approval to the Contract Administrator at least one month prior to commencement of work under this item.

Fabricated wick drain material shall meet the minimum specifications included below.

- 3.1 The Contractor shall submit a 1 m sample of the vertical drain material to the Contract Administrator prior to usage and shall allow two weeks for the Contract Administrator to evaluate the material. The sample shall be stamped or labelled by the manufacturer as being representative of the drain material having the specified trade name. Documentation indicating the source of the drain shall be provided. Approval of the sample by the Contract Administrator shall be required prior to site delivery of the vertical drain material.
- 3.2 Manufacturer certification shall be provided for all drain material delivered to the project.
- 3.3 All drains supplied shall be free of defects, rips, holes or flaws. During shipment the drain shall be protected from damage. During on-site storage the storage area shall be such that the drain is protected from sunlight, dirt, dust, mud, debris and any other detrimental substances.

#### **4.0 Equipment**

- 4.1 Vertical drains shall be installed with equipment, which will minimise disturbance to the granular blanket or the native subsoil during the installation operation. Static or vibratory methods are considered acceptable. Falling weight impact hammers will not be allowed.
- 4.2 The Contractor is advised that the site is considered as an environmentally sensitive area and therefore the control of any water effluent needs to be carefully planned and organized. Jetting techniques, therefore, shall be subjected to the approval of the Contract Administrator.
- 4.3 The Contractor shall be permitted to use augering equipment to predrill or to loosen the native soils and the granular blanket if required to facilitate the installation of the wick drains.
- 4.4 Each prefabricated wick drain shall be installed using a mandrel or sleeve, which shall be advanced through the underlying soil and the granular blanket. The mandrel shall protect the prefabricated drain material from tears, cuts and abrasions during installation and shall be withdrawn after the installation on the drain. The mandrel shall be provided with an "anchor" rod or plate at the bottom to prevent the soil from entering the bottom of the mandrel during installation of the drain and to anchor the bottom of the drain at the required depth at the time of mandrel removal. The projected cross-sectional area of the mandrel and anchor combination shall not exceed 7700 mm<sup>2</sup>.

#### **5.0 Installation**

##### **5.1 Installation Method Proposal Submission**

At least three weeks prior to the installation of the drainage strips, the Contractor shall submit to the Contract Administrator, for review and approval, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- Size, type, weight, maximum pushing force, and configuration of the installation rig.
- Dimensions and length of mandrel.
- Details of drain anchorage.
- Detailed description of proposed installation procedures.
- Proposed methods for overcoming obstructions.
- Proposed methods for splicing drains.

Approval by the Engineer will not relieve the Contractor of his responsibilities to install vertical drain strips in accordance with the plans and specifications.

## 5.2 Construction Sequence

Vertical drains shall be installed subsequent to the construction of the granular blanket and prior to installation of monitoring instruments and placement of the embankment material. The vertical drains shall not be installed in frozen ground and should be protected by a minimum of 2m of earth fill or 4m of rock fill before ground freezing.

## 5.3 Trial Drains

Prior to the installation of prefabricated drains within the areas designated on the plans, the Contractor shall demonstrate to the Quality Verification Engineer that the proposed materials, equipment and installation method produces a satisfactory drain installation in accordance with these specifications. The Contractor will be required to install a total of ten trial drains at locations within the work area as designated by the Contractor Administrator.

Should the ten trial drains be installed to the satisfaction of the Contract Administrator, the trial drains can be incorporated as part of the permanent installation. The Contractor will be compensated for each trial drain if the installation satisfies the requirements of this specification, at the same unit price as the production drains. The Contractor shall not be compensated for unsatisfactory trial drains.

Approval by the Contract Administrator of the method and equipment used to install the trial drains shall not constitute, necessarily, acceptance of the method for the remainder of the project. If, at any time, the Contractor Administrator considers that the method of installation does not produce a drain, which satisfies the project requirements, the Contractor shall alter his method and/or equipment as necessary to comply with these specifications.

## 5.4 Layout

Prefabricated drains shall be located and staked out by the Contractor. The location of the drains shall not vary by more than 150 mm from the locations indicated on the drawings.

## 5.5 Plumbness

Drains shall be installed vertically, within a tolerance of not more than 10 mm per 500 mm. The equipment shall be carefully checked for plumbness, and the Contractor shall provide the Contract Administrator with a suitable means of verifying the plumbness of the mandrel and of determining the depth of the drain at any time.

## **5.6 Splices**

Splices or connections in the vertical drain material shall be done in a professional manner so as to ensure continuity and to avoid any reduction of the flow characteristics of the wick material. Splices shall be a minimum of 150 mm in length.

## **5.7 Cut-off**

The prefabricated drain shall be cut at the surface such that at least a 150 mm length protrudes above the top of the granular blanket at each drain location.

## **5.8 Obstructions**

Where obstructions are encountered below the working surface which cannot be penetrated by the drain installation equipment, the Contractor shall complete the drain from the elevation of the obstruction to the working surface and notify the Contract Administrator. At the direction of the Contract Administrator, the Contractor shall attempt to install a new drain within a 500 mm radius of the obstructed drain. A maximum of two attempts shall be made as directed by the Contract Administrator. The Contractor will be compensated for each obstructed drain unless the drain is improperly completed, in which case no compensation will be allowed.

## **5.9 Preaugering**

It may be necessary to preauger holes through the native soils and the granular blanket to facilitate the installation of the prefabricated wick drain. Any additional cost for preaugering shall be incorporated into the unit price.

## **5.10 Rejected Drains**

Prefabricated drains that are installed beyond the plan location by more than 150 mm, or that are damaged or are not installed in accordance with the specifications described above shall be rejected. Rejected drains may be removed at the Contractor's own expense and time. The Contractor shall not be compensated for the materials and work associated with rejected drains.

Replacement drains shall be installed within a 50 cm radius from the location of the rejected drain as directed by the Contract Administrator.

## **5.11 Geotechnical Instrumentation**

Installation of the drains should be coordinated with the placement of geotechnical instrumentation. Special care should be taken to install drains in such a manner so as not to disturb instrumentation already in place. The replacement of instrumentation damaged as a result of the Contractor's activities will be the responsibility of the Contractor.

## **6.0 Payment**

### **6.1 Measurement of Payment**

Measurement is by Plan Quantity, as may be revised by Adjusted Plan Quantity shall be by the linear metre for all accepted drains installed including the protruding portion. Properly completed obstructed wick drains and properly installed replacement wick drains and trial drains will be measured for payment.

## 6.2 Basis for Payment

Payment at the contract price for the above tender item shall be full compensation for all labour, materials and equipment to complete the work in accordance with the plans and this special provision.

No payment shall be made for unacceptable drains or delays or expenses incurred by the Contractor as a result of improper or unacceptable material or installation.

PRODUCT SPECIFICATIONS			
	TEST METHOD	UNITS	VALUE
<b>PHYSICAL PROPERTIES</b>			
Drain Body Material		Studded or Grooved	Polypropylene
Filter Material		Non-Woven	Polypropylene
Weight	ASTM-D-1777	g/m	75
Width		mm	not less than 100
Thickness	ASTM-D-5199	mm	not less than 3
Mass of Filter	ASTM-D-1777	g/m <sup>2</sup>	130
<b>MECHANICAL PROPERTIES</b>			
Drain Composite Tensile Strength	ASTM D-4595	kN	0.375 @ 10%
Filter Puncture Strength	ASTM-D-751-68	kN	0.335
Filter Grab Strength	ASTM-D-1682	kN	0.8
Filter Trapezoidal Tear	ASTM-D-1117	kN	0.22
Filter Burst Strength	ASTM-D-751-68	kPa	2000
Discharge Capacity @ 70 kPa	ASTM-D4716	m <sup>3</sup> /s	100x10 <sup>-6</sup>
FOS	CAN/CGSB-148.1 No. 10.2	µm	15 to 125
Minimum elongation at break (%)	CAN/CGSB-148.1 No. 7.3	%	15
Water Permeability	ASTM D-4491	m/s	0.000005

## Appendix J

### Comparison of Design Alternatives



This appendix presents a comparison and approximate cost estimates of three design alternatives for the crossing of the proposed Hwy 69 over CNR track. The design alternatives discussed herein are:

#### **COMBINED EPS AND ROCK FILL**

This alternative consists of using EPS for the construction of the embankment within 40m of the north abutment, a transition zone from 40m to 90m to 100m from the abutment where the embankment will consist of a combination of EPS, rock and granular fill. Beyond 90m to 100m of the abutment the embankment will consist of rock fill and granular surcharge. The areas where only EPS fill will be used will also be excavated to 1.5m below the original ground surface and replaced with EPS to further reduce the potential for post-construction time dependent settlements. Wick drains spaced at 1.5m in a triangular pattern will be used within the footprint of the embankment and 5m beyond the embankment toe within 220m of the north abutment to accelerate dissipation of excess pore pressures in the compressible deposits. This option also includes waiting periods of 1.5 and 2.5 years between the end of the embankment construction and removal of surcharge.

#### **EPS FILL**

This alternative includes the use of EPS over the entire length of the north approach embankments, except between Stations 10+670 and 10+700 at the NBL embankment, where the embankment height is 1.2m. The area where EPS fill will be used will also be sub-excavated to 1.5m below the original ground surface and replaced with EPS to further reduce the potential for post-construction time dependent settlements.

#### **BRIDGE EXTENTION**

This alternative includes extending the currently proposed bridge to approximate Station 10+080 in the Dill TWP. The purpose of this alternative is to remove the north approach embankments entirely and practically eliminate the potential for post-construction settlements at this site.

Attached Table J1 presents a summary of the technical aspects and costs associated with each alternative.

**TABLE J1 – DESIGN ALTERNATIVE ANALYSIS**

Alternative	Advantages	Disadvantages	Risks	Anticipated Long-term Settlements (20 years after removal of surcharge)	Costs
Combined EPS and Rock Fill	It has been used in the past by MTO. It is a simple solution at a relatively low cost	Up to 2.5 years of waiting period between end of construction and removal of surcharge is required	Relatively high uncertainty associated with settlement prediction	Within 40m of abutment: <25mm Beyond 40m of abutment: 25mm to 150mm (for 2.5 years of waiting period) or 25mm to 180mm (for waiting period of 1.5 years)	\$4.9 million (*)
EPS Fill	It does not require waiting period. It has been used in the past by MTO. Very fast construction as compared to other options	Costly	This option is based on reducing net increases in stresses in the foundation soils. It poses less risk than the option above.	<25mm throughout the embankment	\$7.6 million (*)
Bridge Extension (SBL: 285m NBL: 260m)	Eliminates uncertainties regarding the performance of the approach embankments	Very high costs. Additional drilling at the pier locations will be required to prove bedrock	Negligible as compared to the other options	Post-construction settlement is not an issue.	\$11.0 million (**)

(\*) Costs do not include contractor's mobilization and pavement costs

(\*\*) The bridge costs provided by TSH